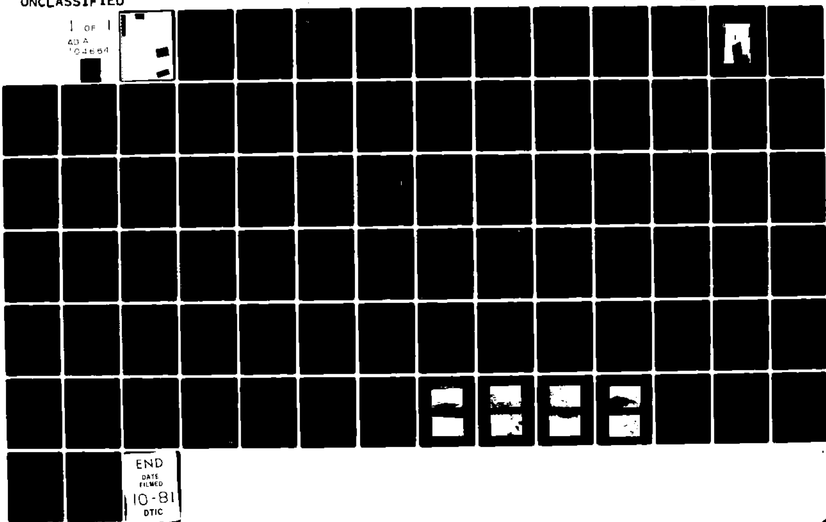
AD-A104 664 HORNER AND SHIFRIN INC ST LOUIS MO F/G 13/13
NATIONAL DAM SAFETY PROGRAM. LAKE WAUWANOKA DAM (MO 30080), MIS--ETC(U)
DEC 78 DACW43-78-C-0143

UNCLASSIFIED NL



AD A104664

LEVEL

UNCLASSIFIED
SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER	2. GOVT ACCESSION NO. AD-A104	3. RECIPIENT'S CATALOG NUMBER 664
4. TITLE (and Subtitle) Phase I Dam Inspection Report National Dam Safety Program Lake Wauwanoka Dam (MO 30080) Jefferson County, Missouri		5. TYPE OF REPORT & PERIOD COVERED Final Report
		6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) Horner & Shifrin, Inc.		8. CONTRACT OR GRANT NUMBER(s) DACW43-78-C-0143
9. PERFORMING ORGANIZATION NAME AND ADDRESS U.S. Army Engineer District, St. Louis Dam Inventory and Inspection Section, LMSED-PD 210 Tucker Blvd., North, St. Louis, Mo. 63101		10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U.S. Army Engineer District, St. Louis Dam Inventory and Inspection Section, LMSED-PD 210 Tucker Blvd., North, St. Louis, Mo. 63101		12. REPORT DATE December 1978
		13. NUMBER OF PAGES Approximately 65
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office) National Dam Safety Program. Lake Wauwanoka Dam (MO 30080), Mississippi - Kaskaskia - St. Louis Basin, Jefferson County, Missouri. Phase I Inspection Report.		15. SECURITY CLASS. (of this report) UNCLASSIFIED
		15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Dam Safety, Lake, Dam Inspection, Private Dams

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report was prepared under the National Program of Inspection of Non-Federal Dams. This report assesses the general condition of the dam with respect to safety, based on available data and on visual inspection, to determine if the dam poses hazards to human life or property.

DD FORM 1 JAN 73 1473 EDITION OF 1 NOV 65 IS OBSOLETE

UNCLASSIFIED
SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

DEPARTMENT OF THE ARMY
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
210 NORTH 12TH STREET
ST. LOUIS, MISSOURI 63101

IN REPLY REFER TO

SUBJECT: Lake Wauwanoka Dam Phase I Inspection Report

This report presents the results of a field inspection and an evaluation of the Lake Wauwanoka Dam.

It was prepared under the National Program of Inspection of Non-Federal Dams.

This dam has been classified as unsafe, non-emergency by the St. Louis District as a result of the application of the following criteria:

1) Spillway will not pass 50 percent of the Probable Maximum Flood.
2) Overtopping could result in dam failure.
3) Dam failure significantly increases the hazard to loss of life downstream.

SUBMITTED BY: SIGNED — Chief, Engineering Division — 6 DEC 1978 — Date

APPROVED BY: SIGNED — Colonel, CE, District Engineer — 6 DEC 1978 — Date

LAKE WAUWANOKA DAM
JEFFERSON COUNTY, MISSOURI

MISSOURI INVENTORY NO. 30080

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

PREPARED BY:

HORNER & SHIFRIN, INC.
5200 OAKLAND AVENUE
ST. LOUIS, MISSOURI 63110

FOR:

U.S. ARMY ENGINEER DISTRICT, ST. LOUIS
CORPS OF ENG?NEERS

NOVEMBER 1978

HS-7848

PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam:	Lake Wauwanoka Dam
State Located:	Missouri
County Located:	Jefferson
Stream:	Dry Creek
Date of Inspection:	27 July 1978

The Lake Wauwanoka Dam was visually inspected by engineering personnel of the office of Horner & Shifrin, Inc., Consulting Engineers, St. Louis, Missouri. The purpose of the inspection was to assess the general condition of the dam with respect to safety and, based upon this inspection and available data, determine if the dam poses a hazard to human life or property.

The following summarizes the findings of the inspection and the results of certain hydrologic/hydraulic investigations performed under the direction of the inspection team.

Based on a visual inspection, the present general physical condition of the dam is considered to be satisfactory; however, the following deficiencies were noticed during the inspection and are considered to have an adverse effect on the overall safety and future operation of the dam:

1. The upstream face of the dam at the waterline has a grass cover, except for a section about 300 feet long near the center of the dam where stone riprap has been placed to protect it from erosion. A grass covered slope is not considered adequate to protect the slope from erosion by wave action or from fluctuations of the water level.

2. A dense cover of vegetation that may contain animal burrows and numerous small trees exist on the downstream face of the dam as well as the area adjacent to the downstream toe of slope. Tree roots and animal burrows can provide passageways for seepage that may develop into a piping condition.

3. The pipe subdrain, installed to collect seepage and located in the area adjacent to the downstream toe of slope near the left (looking downstream) abutment, is in poor condition for a portion of its length. A section of collector pipe is lying exposed on the ground surface. The pipe is broken with water flowing from the broken end and ponding on the area in the vicinity of the pipe. Cattails are present along the route of the pipe in this area.

4. The left bank (dam side) of the spillway at the lake approach channel and through the control section is protected from erosion by a grass covered slope. A grass covered slope is not considered adequate to prevent erosion of the bank by high spillway flows.

5. Numerous small trees exist in the spillway exit channel just below the crest section. The presence of these trees will obstruct the flow and could cause spillway discharge to overflow the channel and flood the adjacent embankment and the area adjacent to the downstream toe of slope. Flooding of the area adjacent to the dam may impair the stability of the dam.

The crest of the dam was found to be approximately 1.5 feet lower at a location near the center of the dam than the crest of the dam in the area adjacent to the spillway. As a result of this low top of dam section, the capacity of the spillway to discharge lake outflow without overtopping the dam is reduced considerably. According to the criteria set forth in the recommended guidelines (see text) the minimum spillway design flood for this dam, which is classified as intermediate in size and of high hazard potential, is specified to be the Probable Maximum Flood (PMF). PMF is the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region. Results of a hydrologic/hydraulic analysis indicate that the existing spillway is inadequate to pass lake outflow resulting from a storm of PMF magnitude. The spillway is adequate to pass the lake outflow resulting from the 1 percent chance (100-year frequency) flood. The existing spillway is capable of

passing lake outflow corresponding to about 27 percent of the PMF. The length of the downstream damage zone, should failure of the dam occur, is estimated to be six miles. Within the possible damage zone are five homes and three improved road bridges.

A review of available data did not disclose that seepage and stability analyses of the dam were performed. This is considered a deficiency and should be rectified.

It is recommended that the Owner take the necessary action, without delay, to correct the safety defects and deficiencies reported herein.

Albert B. Becker, Jr.

Albert B. Becker, Jr.
P.E. Missouri E-9168

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM
LAKE WAUWANOKA DAM - ID NO. 30080

TABLE OF CONTENTS

TABLE OF CONTENTS - Continued

LIST OF PLATES AND CHARTS

APPENDIX

OVERVIEW OF LAKE AND DAM

PHASE I INSPECTION REPORT
NATIONAL DAM SAFETY PROGRAM

LAKE WAUWANOKA DAM - ID NO. 30080

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL

a. Authority. National Dam Inspection Act, Public Law 92-367, dated 8 August 1972.

b. Purpose of Inspection. The purpose of this visual inspection was to make an assessment of the general condition of the dam with respect to safety and, based upon available data and this inspection, determine if the dam poses a hazard to human life or property.

c. Evaluation Criteria. This evaluation was performed in accordance with the "Phase I" investigation procedures as prescribed in "Recommended Guidelines for Safety Inspection of Dams," Appendix D to "Report of the Chief of Engineers on the National Program of Dams," dated May 1975.

1.2 DESCRIPTION OF PROJECT

a. Description of Dam and Appurtenances. The Wauwanoka Lake Dam is an earthfill type embankment constructed across a narrow valley in the northeastern part of the Ozarks and rising approximately 60 feet above the original stream bed. Topography adjacent to the valley is rolling to steep. In general, the mantle is a stoney, red clay soil overlying limestone bedrock.

Lake level is governed by a spillway section cut into bedrock and located adjacent to the right (looking downstream) abutment. The control section of the spillway consists of a concrete wall with a gated opening near its center.

Behind the spillway a trench has been excavated in rock to provide an outlet for the release of lake water when it is necessary to lower the lake below normal pool. Below the spillway crest the outlet channel consists of a series of rock falls that leads to the valley floor and the original stream, Dry Creek. An 8-inch sanitary sewer, that serves the residential development that surrounds the lake, crosses the spillway channel (the pipe line is encased in the concrete weir section) and runs parallel to the dam. The sewer is located in the downstream slope just below the dam crest at a depth of about 4 feet. At a point near the left abutment the sewer line turns eastward and proceeds for about 0.3 of a mile to a treatment plant. At normal pool elevation the lake occupies approximately 86 acres. A manually operated slide gate, located in the concrete weir at the spillway, is capable of lowering the lake about 4 feet below the normal pool level. A plan of the Lake Wauwanoka Subdivision, showing the dam, is shown on Plate 2.

b. Location. The dam and lake are located on Dry Creek, approximately 2 miles east of Hillsboro, Missouri, in Jefferson County, as shown on the Regional Vicinity Map, Plate 1. The dam is located in Section 1, Township 40 North, Range 4 East, approximately 2 miles southeast of the intersection of State Highway 21 and State Route A.

c. Size Classification. The size classification, based on the height of the dam and storage capacity, is categorized as intermediate. (Per Table 1, Recommended Guidelines for Safety Inspection of Dams.)

d. Hazard Classification. Lake Wauwanoka Dam, according to the St. Louis District, Corps of Engineers, has a high hazard potential, meaning that if the dam should fail, there may be loss of life, serious damage to homes, extensive agricultural, industrial and commercial facilities, important public utilities, main highways, or railroads. The estimated flood damage zone, should failure of the dam occur, as determined by the St. Louis District, extends six miles downstream of the dam. Within the possible damage zone are five homes and three improved road bridges.

e. Ownership. The lake and dam are owned by Lake Wauwanoka, Inc., a corporation comprised of approximately 100 members who are property owners at Lake Wauwanoka. The remaining property owners, about 290 in number, are not members of the corporation. The current chairman of the corporation, Mr. John F. Buehler, resides at Lake Wauwanoka and the address is Route 5, Box 199, Hillsboro, Missouri, 63050.

f. Purpose of Dam. The dam impounds water for recreational use by the surrounding residential property owners.

g. Design and Construction History. The dam was constructed sometime during the early 1940's. Based on the report by Brucker & Thacker on the investigations of subsurface conditions (see Charts 2-1 thru 2-9), the builder of the dam was a Mr. Schielly from Kansas City, Missouri. Mr. Schielly's present location and status are unknown.

In 1967 the firm of Brucker & Thacker, Consulting Engineers, Brentwood, Missouri, was retained to investigate the cause of lake water leaking through the dam and to make recommendations for sealing the dam to prevent the loss of water. At that time, the lake was estimated to be losing water at a rate of about 5 million gallons per day (3,500± gpm). According to the Owner's representative, earlier attempts to seal the leak by pressure grouting the suspected permeable areas with asphalt were unsuccessful. Subsequently, under the direction of Brucker & Thacker, a grouting program was undertaken and the loss of water from the lake was virtually halted. At present, measurement of lake seepage appears to be on the order of 4 gpm.

According to the Owner's representative, a sanitary sewer system serving the homes that surround the lake was designed by Mr. John C. Pritchard, now deceased, and installed about 1953. The Owner's representative reports that there have been no serious maintenance problems associated with the sewer system since its construction approximately 25 years ago.

In 1976, the Owner improved the spillway by constructing a new concrete weir section across the channel. A slide gate was installed in the weir for the purpose of partially dewatering the lake.

In 1976, the Owner installed a subdrain system to collect and drain seepage that appeared in the vicinity of the left (looking downstream) abutment. The owner has also installed an aeration system in the lake for the purpose of controlling aquatic vegetation, and at least one pond for the purpose of preventing silt from washing into the lake. At present there is also a program underway to place stone riprap across the upstream face of the dam in order to prevent erosion.

h. Normal Operational Procedure. The lake level is unregulated.

1.3 PERTINENT DATA

a. Drainage Area. The area tributary to the lake, with the exception of the residential property surrounding the lake and the eastern suburbs of the City of Hillsboro, is undeveloped with a portion covered by timber and the remainder used for agricultural purposes. The watershed above the dam amounts to approximately 1,320 acres. The watershed area is outlined on Plate 1.

b. Discharge at Damsite.

(1) Estimated known maximum flood at damsite ... 300 cfs[(1)]

(2) Spillway capacity ... 2,000 cfs

c. Elevation (ft. above MSL). The top of the concrete weir at the center of the gate, located at the spillway control section, was assumed to be elevation 612, the basis for this assumption being the elevation for the lake surface shown on the 1960 DeSoto, Missouri, Quadrangle Map, 7.5 minute series.

(1) Discharge value over the spillway computed for water surface at elevation 613.5, the high lake level since completion of new weir in 1976 as reported by a representative of the Owner living adjacent to the lake.

(1) Top of dam ... 616.6 (min.)

(2) Normal pool (spillway crest) ... 612.0

(3) Streambed at centerline of dam ... 557±

(4) Maximum tailwater ... Unknown

d. Reservoir.

(1) Length at normal pool (elevation 612.0) ... 5,100 ft.

(2) Length at maximum pool (elevation 616.6) ... 5,400 ft.

e. Storage.

(1) Normal pool ... 2,370 ac.ft.

(2) Top of dam (incremental) ... 430 ac.ft.

f. Reservoir Surface.

(1) Top of dam ... 99 acres

(2) Normal pool ... 86 acres

g. Dam.

(1) Type ... Earthfill, clay core[(1)]

(2) Length ... 1,045 ft.

(3) Height ... 60 ft.

(4) Top width ... 14 ft.

(5) Side slopes

  a. Upstream ... 1v on 3h

  b. Downstream ... 1v on 2h

(6) Cutoff ... Clay core[(1)]

(7) Slope protection

  a. Upstream ... Grass, riprap (at center)

  b. Downstream ... Grass

h. Spillway.

(1) Type ... Concrete, unregulated

(2) Length of weir ... 53 ft.

(1) According to quotation appearing in report by Brucker & Thacker, Consulting Engineers.

(3) Crest elevation ... 612.0

(4) Approach channel ... Lake

(5) Control section ... Rock cut

(6) Exit channel ... Rock falls

h. Gate for Lake Drawdown.

(1) Type ... Slide, stainless steel

(2) Operation ... Manual

(3) Size ... 24-inch wide by 30-inch high

(4) Invert elevation ... 608.0±

(5) Location ... Spillway weir, 20 ft. from left end of weir

SECTION 2 - ENGINEERING DATA

2.1 DESIGN

With the exception of the profile and cross sections of the dam as indicated in paragraph 2.2, no engineering data relating to the design of the dam, the hydrology of the watershed, or the hydraulics of the spillway are known to exist.

2.2 CONSTRUCTION

The dam was constructed sometime during the early 1940's. According to drawings provided by the Owner, a profile, showing the dam crest and the original ground line and cross sections of the embankment, at selected intervals of about 100 feet on centers, were developed by the designer for use in constructing the dam. A review of these drawings indicated the planned top width to be 14 feet, the upstream slope ratio to be 1v on 3h, the downstream slope ratio to be 1v on 2h, and the maximum height of dam at the centerline of the structure to be about 60 feet. Fill quantities were also indicated on the sections. Elevations were based on an arbitrary datum and not related to U.S.G.S. datum.

2.3 OPERATION

The lake level is governed by overflow of an uncontrolled concrete weir type spillway. A representative of the Owner reported that a fence crossing the spillway control section, to prevent fish from being washed from the lake, was removed several years ago after experiencing a flood that nearly overtopped the dam because debris accumulated on the fence restricting spillway discharge and creating backwater that raised the lake level.

Since removal of the fence across the spillway the maximum known loading on the dam, according to a resident living adjacent to the lake, was a storm that produced a rise of about 18 inches above normal pool level.

In 1967 the firm of Brucker & Thacker, Consulting Engineers, investigated the source of a leak that was occurring at a point near the center of the dam. Three test holes (TH-1, TH-2 and TH-3) were drilled along the downstream toe of slope and a fourth hole (TH-4) was drilled at the crest near the center of the dam for the purpose of determining subsurface conditions. The locations of these test holes are shown in plan on Plate 3 and the boring logs are shown on Plate 5. Geophysical investigations using electrical resistivity were also performed to aid in evaluating the subsurface conditions in order to detect the source of the leak. The geophysical investigations were performed by Dr. Richard D. Rechtein, Geophysical Consultant, Rolla, Missouri, for Brucker & Thacker. A profile of the dam showing the location of the resistivity test points and Test Holes 1, 2, and 3 is shown on Plate 4. A subsurface profile based on an interpretation of the resistivity tests correlated with the test borings is shown on Plate 8. Based on these subsurface conditions, it was concluded that a highly permeable material (gravel) existed in the vicinity of Test Point G-4, or about 350 feet north of the spillway. A plan for sealing the area of the dam where the zone of permeable material was detected was prepared by Brucker & Thacker and, shortly thereafter (about September of 1967), a program of pressure grouting the area was carried out. The plan location of grout holes and quantities of grout injected into these holes is shown on Plates 6 and 7, respectively. A report prepared by Brucker & Thacker concerning their investigations and conclusions is included on Charts 2-1 through 2-9. A report (letter) by Dr. Rechtein covering the initial geophysical investigations performed under his direction is presented on Charts 2-10 through 2-12. A follow-up geophysical investigation, reference Charts 2-13 and 2-14, was also performed by Dr. Rechtein during October of 1967 after completion of the grouting program. A subsurface profile on the upstream side of the dam and an electrical resistivity depth profile based on this study are presented on Plate 9. This investigation indicated that, in the locations tested along the upstream face of the dam, there were no regions of significant seepage.

2.4 EVALUATION

a. Availability. Engineering data for assessing the design of the dam and spillway were not available.

b. Adequacy. No design data available. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency. These seepage and stability analyses should be performed for appropriate loading conditions and made a matter of record.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS

a. General. A visual inspection of the dam and spillway was made by Horner & Shifrin engineering personnel on 27 July 1978. Also inspected was the area downstream from the dam including the various road stream crossings between the dam and Joachim Creek. Photographs of the dam and spillway, taken at the time of the inspection, are included on Pages A-1 through A-4 of the Appendix.

b. Dam. The visible portions of the upstream and downstream slopes (see Photos 1 and 2) of the dam appeared to be in satisfactory condition, although numerous small trees and dense vegetation were present on the downstream face as well as the area adjacent to the dam. Riprap, consisting of crushed limestone rock about 2- to 3-inches in size, serves to protect the upstream face of the dam above and below the normal waterline across a section of the dam near the center. The remainder of the upstream face above the waterline is grass covered. No slides, sloughing, or cracking of the embankment was noticed. No holes or animal burrows were noticed in the dam slope or elsewhere. However, due to the dense vegetation covering the downstream face, it could not be concluded that none exist.

A broken subdrain pipe (see Photo 3), located in the area adjacent to the downstream toe of slope near the left abutment, was allowing seepage water to escape and pond (see Photo 5) in the vicinity of the break. The ground in the area along the route of the subdrain was noticeably soft and wet with cattails (see Photo 6) growing in several locations. It could not be determined if all of the flow escaping the broken pipe was recollected by the subdrain. At the outlet end of the collector pipe (see Photo 4) the flow was measured to be about 4 gpm.

The ground surface in the area adjacent to the downstream toe of slope at a point about 350 feet north of the spillway was covered with asphalt over an area totaling approximately 400 square feet. This asphalt is believed to be part of the bituminous material used for grouting during June and July of 1967, when an attempt was made to prevent loss of water from the lake by sealing the permeable zone in this vicinity of the dam with asphalt.

Three sanitary sewer manholes, one at each end and one near the center of the dam, are present on the downstream face just below the dam crest. Heavy concrete slabs without access lids cover the manholes and prevented inspection of the interior.

The elevation of the dam crest, as determined by survey, was found to be approximately 1.5 feet lower across the central area of the dam than the top of dam in the area adjacent to the spillway. A plot of the 1967 Brucker & Thacker survey data also indicated the top of the dam to be 1.5 feet lower at a location near the center than the top of the dam in the area near the spillway. Due to the fact that the two surveys (1967 and 1978) were made using different elevation datum, exact correlation of the top of dam levels, in order to determine interim settlement, could not be made. A profile of the dam crest centerline extending through the spillway section, based on survey data obtained during the inspection, is shown on Plate 9.

c. Spillway. The concrete spillway weir (see Photo 7) and stainless steel slide gate appeared to be in good condition. No deterioration of the concrete due to weathering or damage from ice was noticed. At the time of the inspection, the lake level was about 0.1 foot above the weir at the gate location. The flow passing the weir was concentrated in a trench cut in the rock (limestone) floor of the control section. The trench was found to be approximately 6 feet wide, 4 feet deep, and 120 feet long at the control section. The gate at the concrete weir is located in lime with the trench. Through the control section, the right bank is in rock cut while the left bank, dam, (see Photo 7) consists of earthfill with a turf cover. The exit channel immediately below the control section consists of a series of rock falls with numerous

small trees (see Photo 8) growing in several locations. A profile of the spillway channel from the weir to a point downstream of the control section is shown on Plate 9.

d. Downstream Channel. The downstream channel is unimproved. Dry Creek joins Little Creek about 2 miles below the dam and Little Creek joins Joachim Creek about 6 miles below the dam.

e. Reservoir. The area surrounding the lake is nearly entirely occupied by homes and other improvements. Concrete walls serve to protect the shore line at many locations. A beach for swimming is located at the upstream end of the lake. An aeration system for controlling aquatic growth, consisting of four small electrically operated compressors with plastic tubing air lines that distribute air to various areas of the lake, is located about the lake. A small pond, located just upstream of a cove on the north side of the lake, serves to prevent siltation of this arm of the lake. According to a representative of the Owner, there is no appreciable sediment within the lake at the present time.

3.2 EVALUATION

The deficiencies observed during this inspection are not considered significant to warrant immediate remedial action.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES

The spillway is uncontrolled. The water surface level is governed by rainfall runoff, evaporation, seepage, and the capacity of the uncontrolled spillway.

4.2 MAINTENANCE OF DAM AND SPILLWAY

According to the Owner's representative, the grass on the dam crest and the upstream face of the dam is mowed frequently throughout the growing season. Riprap, consisting of 2- to 3- inch size limestone, is periodically being placed across the upstream face of the dam and, to date, about 30 percent of the dam has been covered. Vegetation on the downstream slope and the area adjacent to the toe of dam is sprayed yearly to control weed growth. The entire downstream area was cleared of trees and brush in 1976. The Owner's representative reported that the dam has been seeded with K-31 Fescue grass on several occasions and that the grass covered areas are fertilized once a year.

The subdrain system located near the left abutment was installed in 1976 in an attempt to drain and dry out the swampy areas below the north end of the dam. Further, the Owner's representative reported that when the lake was lowered about 3.5 feet in 1977, the rate of flow as measured at the outlet end of the subdrain pipe was reduced from approximately 3.6 gpm to 1.5 gpm.

A new concrete weir, including a gate to partially dewater the lake, was installed across the spillway in 1976 replacing a deteriorated concrete and masonry structure.

An inspection of the dam was made by the Soil Conservation Service (SCS) in 1974. This inspection indicated the existence of many minor deficiencies. These deficiencies, along with recommendations for remedial work to resolve these problems, are presented in their letter of 12 April 1974 and are included herein as reference Charts 4-1 through 4-3.

4.3 MAINTENANCE OF OUTLET OPERATING FACILITIES

No spillway control facilities exist at this dam, with the exception of the 24-inch wide by 30-inch high slide gate located at the spillway weir and used to partially dewater the lake. Since this gate is fabricated of stainless steel, little maintenance is required.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT

The inspection did not reveal the existence of a dam failure warning system.

4.5 EVALUATION

Based on the general condition of the dam and spillway, as well as the interest and concern shown by the Owner's representative in charge of dam maintenance, it is evident that the dam and appurtenant structures are, for the most part, well maintained. It is recommended, however, that the grass and vegetation on the downstream slope be cut periodically during the growing season.

SECTION 5 - HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. Design Data. Design data is not available.

b. Experience Data. The drainage area and lake surface area were measured from the USGS DeSoto, Missouri, Quadrangle Map. The proportions and dimensions of the spillway and dam were determined by surveys made during the inspection.

c. Visual Observations.

(1) The concrete weir spillway crest and the excavated rock outlet channel and rock falls are in good condition. Numerous small trees exist in some areas of the exit channel.

(2) A 2-foot wide by 3-foot high stainless steel slide gate in the concrete weir section and an excavated trench, 6± feet wide by 4± feet high, through the rock outlet channel section are provided to partially dewater (to elevation 608.0±) the lake.

(3) The spillway and outlet channel are located in the right abutment of the dam. Spillway releases within the capacity of the spillway section will not endanger the integrity of the dam.

d. Overtopping Potential. The spillway section is not adequate to pass the probable maximum flood or the 1/2 probable maximum flood, but will pass the 1 percent chance (100-year frequency) flood without overtopping the dam. The results of a dam overtopping analysis are as follows:

Ratio of PMF	Q - Peak Outflow (cfs)	Max. Lake Water Surface Elevation	Maximum Depth of Flow Over Dam (Elev. 616.6)	Duration of Overtopping of Dam (Hours)
0.27	2,000	616.6	0	0
0.50	6,500	618.5	1.9	2.9
1.0	17,300	620.3	3.7	5.9
100-Year Flood	1,540	615.9	0	0

The flow safely passing the spillway just prior to overtopping amounts to about 2,000 cfs, which is the outflow corresponding to about 27 percent of the probable maximum flood inflow and exceeds the outflow from the 1 percent chance (100-year frequency) flood.

Procedures and data for determining the probable maximum flood, the 100-year frequency flood, and the discharge rating curve for flow over the spillway and the dam crest are presented on Pages B-1 and B-2 of the Appendix. A listing of the HEC-1DB input data is shown on Pages B-3 through B-5 of the Appendix.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY

a. Visual Observations. Visual observations which adversely affect the structural stability of the dam are discussed in Section 3, paragraph 3.1b.

b. Design and Construction Data. No design and construction data relating to the structural stability of the dam are known to exist.

c. Operating Records. No appurtenant structures or facilities requiring operation exist at this dam. The Owner has monitored seepage (flow rate measured at outlet of subdrain pipe) since installation of the collector pipe in 1976, and has also determined the relative elevations of the dam and spillway crests by survey. Measured seepage as indicated was found to be on the order of 3.6 gpm with the lake at normal pool level, and the dam crest was determined to be lower at the center than at the south (spillway) or north ends. Both determinations were verified at the time of the visual inspection.

d. Post Construction Changes. With the exceptions of the grouting done in 1967 under the direction of Brucker & Thacker, Consulting Engineers, the sanitary sewer installed in the downstream slope just below the crest and paralleling the dam, and the subdrain installed in the area below the dam near the left abutment, no post construction changes were made which could affect the structural stability of the dam, according to the Owner's representative.

e. Seismic Stability. Since the dam is located within a Zone II seismic probability area, an earthquake of the magnitude predicted is not expected to produce a hazardous condition to the dam, provided that static stability conditions are satisfactory and conventional safety margins exist.

SECTION 7 - ASSESSMENT/REMEDIAL MEASURES

7.1 DAM ASSESSMENT

a. Safety. A hydraulic analysis indicated the spillway to be capable of passing lake outflow of about 2,000 cfs without the level of the lake exceeding the low point in the dam. A hydrologic analysis of the lake watershed area, as discussed in Section 5, indicated that for storm runoff of probable maximum flood magnitude the lake outflow would be on the order of 17,300 cfs, and for the 1 percent chance (100-year frequency) flood the lake outflow would be approximately 1,540 cfs.

No stability or seepage analyses of the dam, nor hydraulic analyses of the spillway, are known to exist.

b. Adequacy of Information. Due to the lack of sufficient detail engineering design and construction data, the assessments reported herein were based largely on external conditions as determined during the visual inspection. The data and reports prepared by Brucker & Thacker, Consulting Engineers, and by Dr. Rechtien, Geophysical Consultant, were used in evaluating the condition of the dam with regard to seepage. Those recommendations with regard to the hydrology of the watershed and the capacity of the spillway were based on a hydraulic/hydrologic study. Seepage and stability analyses comparable to the requirements of the "Recommended Guidelines for Safety Inspection of Dams" were not available, which is considered a deficiency.

c. Urgency. The items concerning the safety of the dam noted in paragraph 7.1a and the remedial measures recommended in paragraph 7.2 should be accomplished in the near future.

d. Necessity for Phase II. Based on the results of the Phase I inspection, a Phase II investigation is not recommended.

e. Seismic Stability. Since the dam is located within a Zone II seismic probability area, an earthquake of the magnitude predicted is not expected to produce a hazardous condition to the dam, provided that static stability conditions are satisfactory and conventional safety margins exist.

7.2 REMEDIAL MEASURES

a. Recommendations. The following actions are recommended:

(1) Based upon criteria set forth in the recommended guidelines, alterations to the design of the dam should be made in order to pass lake outflow resulting from a storm of probable maximum flood magnitude. It is recommended, in any event, that the low area located near the center of the dam be raised so that it does not govern spillway capacity.

(2) Obtain the necessary soil data and perform stability and seepage analyses in order to determine the structural stability of the dam for all operational conditions. Seepage and stability analyses should be performed by a professional engineer experienced in the design and construction of dams.

b. O & M Maintenance and Procedures. The following O & M maintenance and procedures are recommended:

(1) Provide slope protection across the remaining portions of the upstream face of the dam above and below the normal waterline in order to prevent erosion by wave action or fluctuation in lake level. If stone riprap is to be used, it is recommended that the smaller (2- to 3-inch size) stone be covered with a layer of larger size stone in order to insure stability of the smaller size course.

(2) Remove the small trees and dense vegetation from the downstream face and the area adjacent to the dam. The existing turf cover should be restored

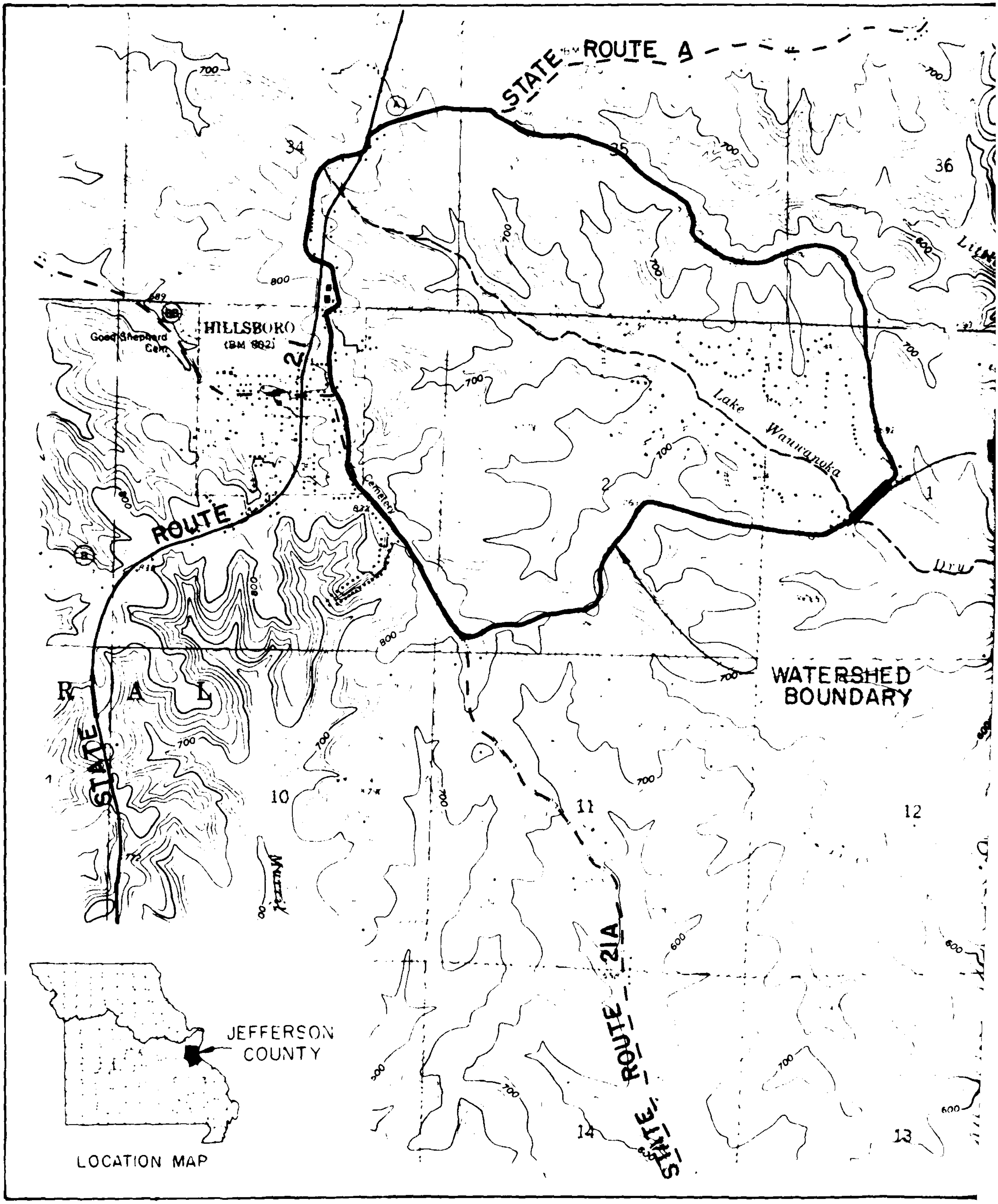
if destroyed or missing. Maintain the turf cover on the slope and below the dam at a height that will not hinder inspection or harbor burrowing animals. Voids created by burrowing animals and tree roots can provide pathways for seepage and the possibility of piping.

(3) Restore the pipe subdrain located in the area adjacent to and below the left abutment in order to prevent loss of water from the collector pipe and subsequent softening of the ground in the vicinity of the pipe. Wet, soft ground can impair the stability of the dam. The collector pipe should be installed such that it is not subject to damage by maintenance equipment.

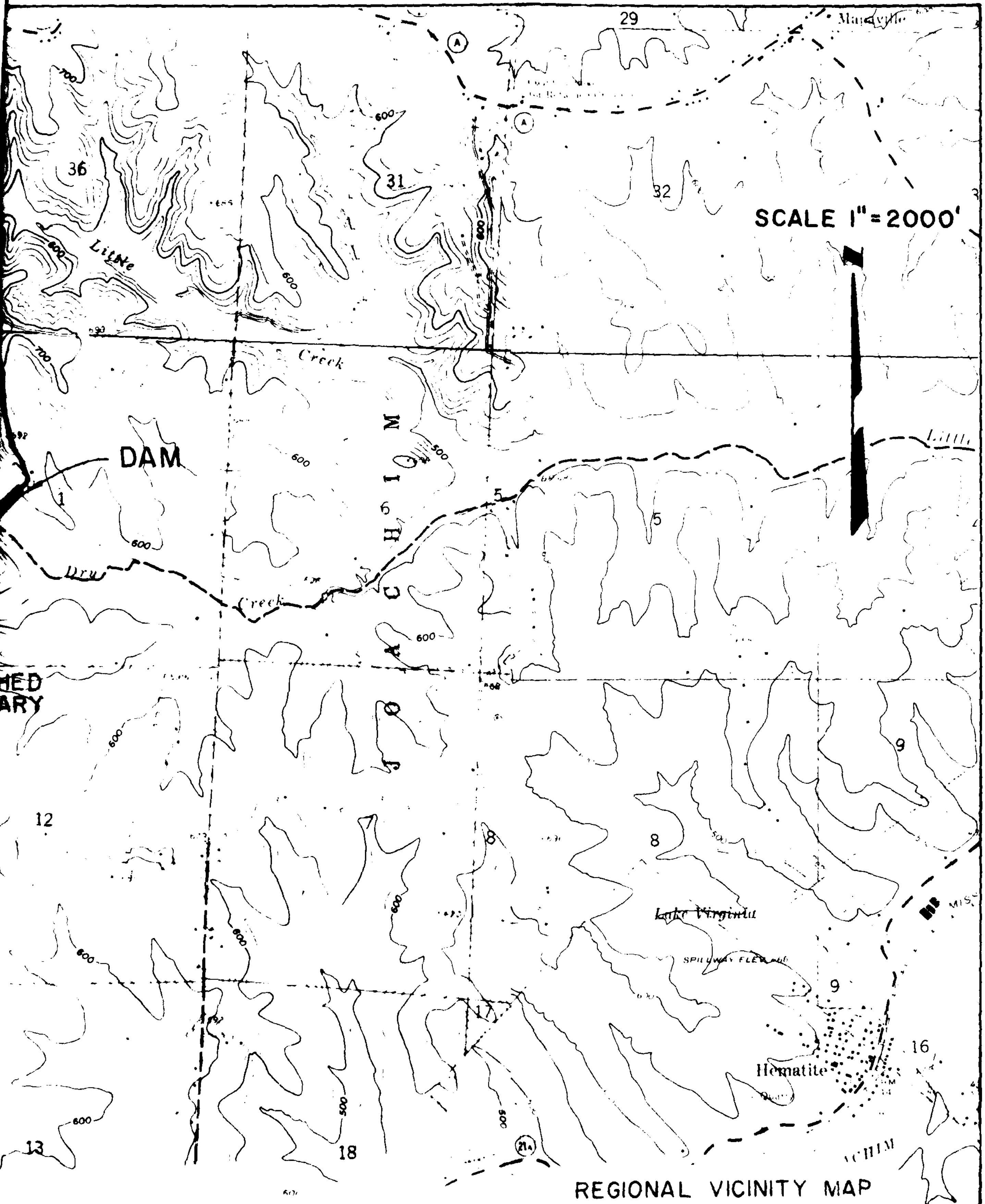
(4) Provide some form of slope protection at the left bank of the spillway in the area adjacent to the dam. Lake outflow corresponding to the recommended spillway design flood or lesser floods could severely erode this grass covered bank.

(5) Remove the trees from the spillway exit channel section in order to allow flow to reach the downstream channel unrestricted. Restricting spillway discharge can result in flooding of the area adjacent to the downstream toe of dam and conditions detrimental to the stability of the embankment.

(6) A detailed inspection of the dam should be instituted on a regular basis by an engineer experienced in the design and construction of earthfill type dams. It is also recommended, for future reference, that records be kept of all inspections and remedial measures.



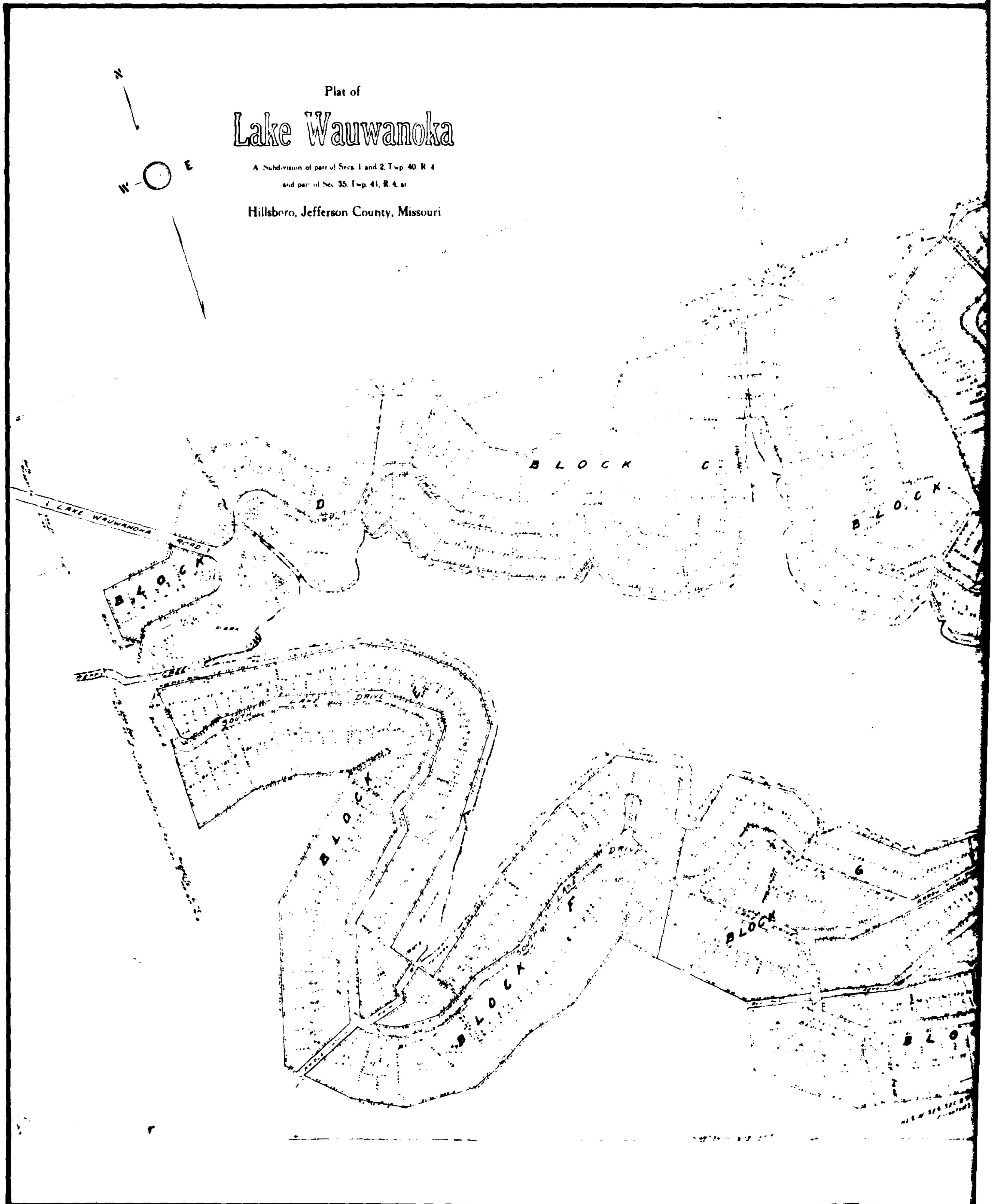
STATE ROUTE A
HILLSBORO
(BM 802)
Good Shepherd Cem.
ROUTE 21
Lake Wauwanoka
Cemetery
WATERSHED BOUNDARY
STATE ROUTE
STATE ROUTE 21A
JEFFERSON COUNTY
LOCATION MAP
34
35
36
2
1
10
11
12
14
13
700
800
600

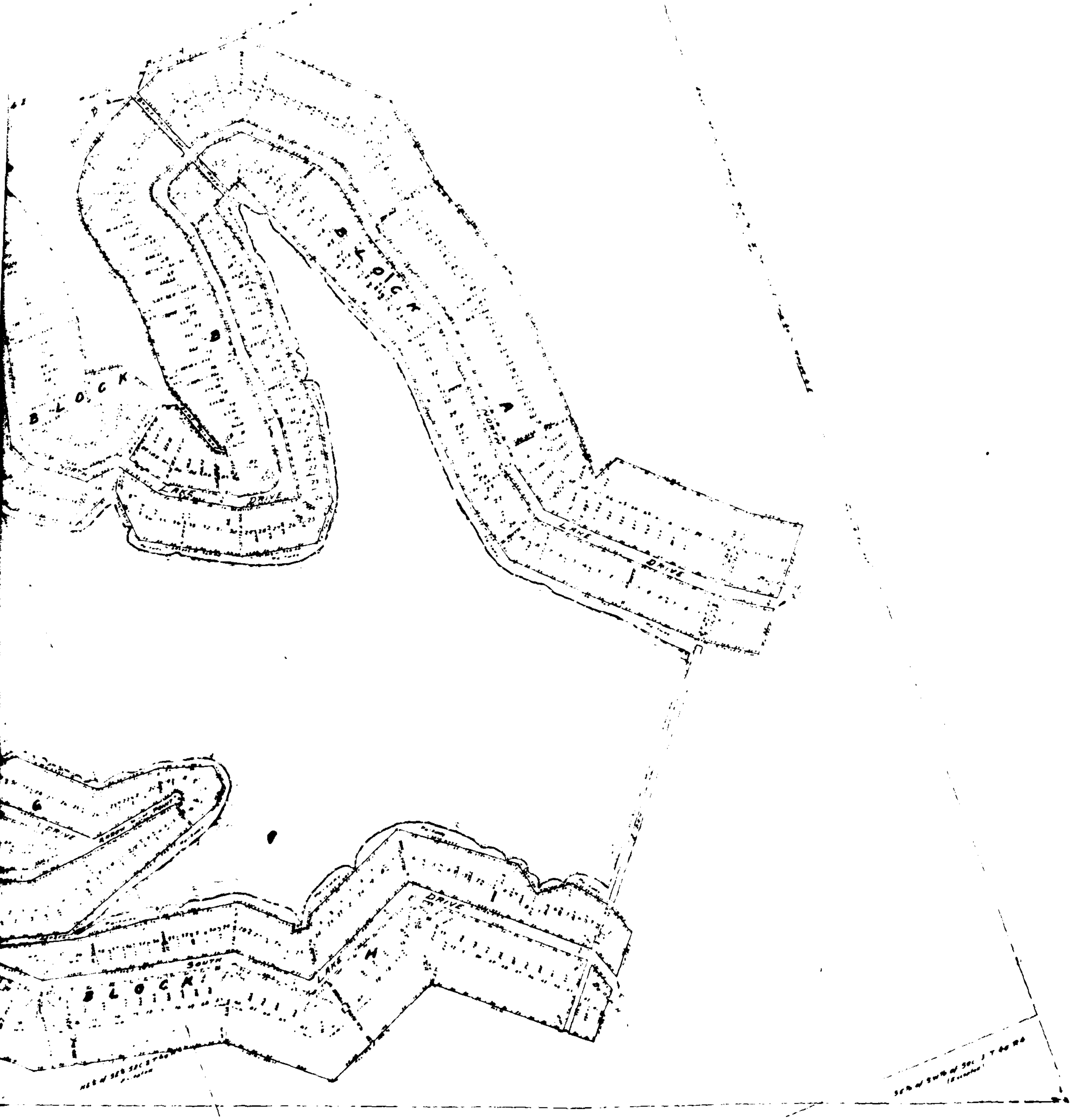


SCALE 1" = 2000'
DAM
Little
Creek
Dry
Creek
JOACHIM
Lake Virginia
SPILLWAY
Hematite
HED
ARY
29
36
31
32
1
6
5
5
12
7
8
8
9
9
13
18
17
16
600
700
500
REGIONAL VICINITY MAP

PLATE I

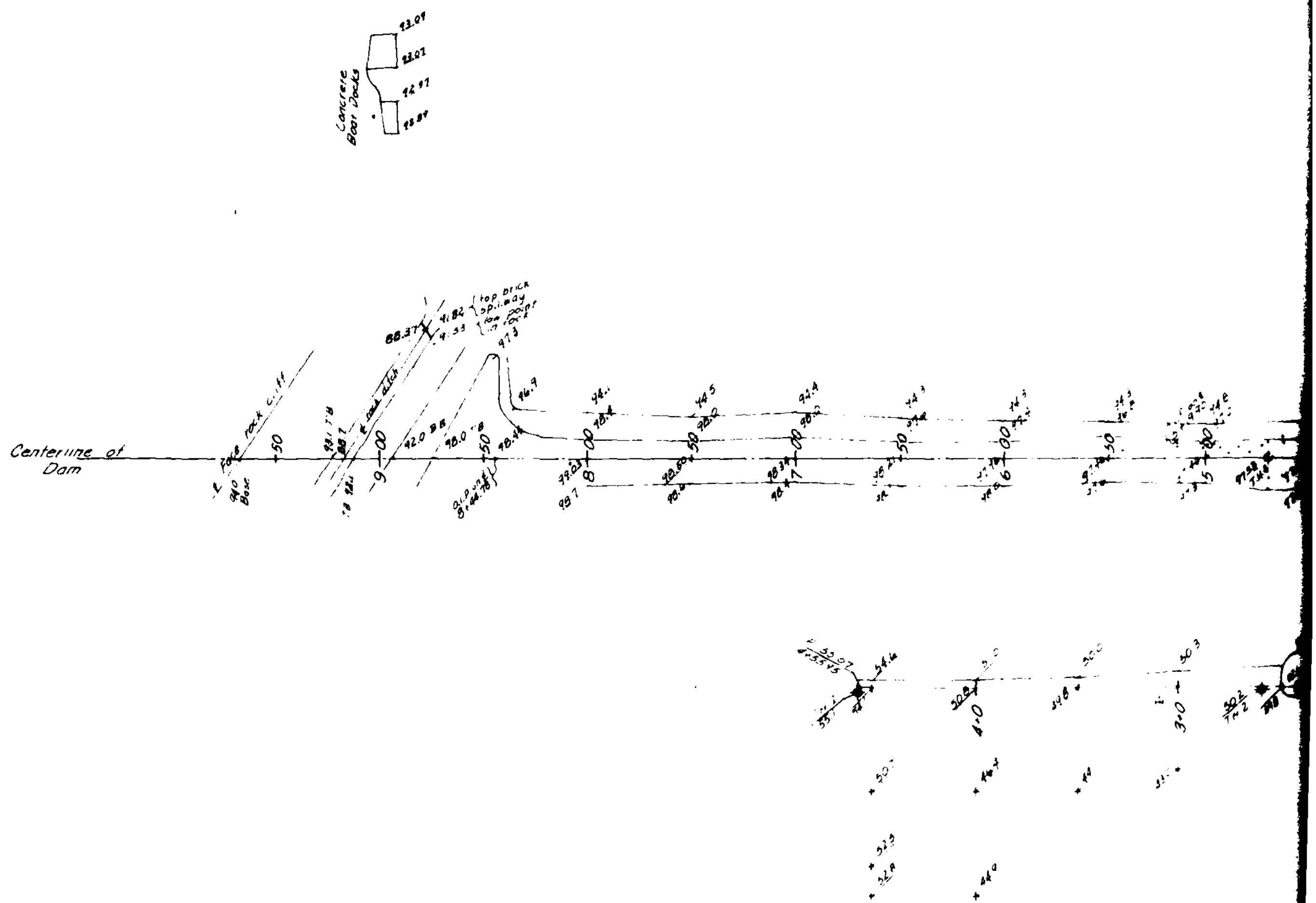
12


N
W
E
Plat of
Lake Wauwanoka
A Subdivision of part of Secs. 1 and 2, Twp. 40, R. 4
and part of Sec. 35, Twp. 41, R. 4, at
Hillsboro, Jefferson County, Missouri
BLOCK C
BLOCK
D
BLOCK
LAKE WAUWANOKA ROAD
SOUTH LAKE DRIVE
E
BLOCK
F
BLOCK
G
BLOCK
BLO

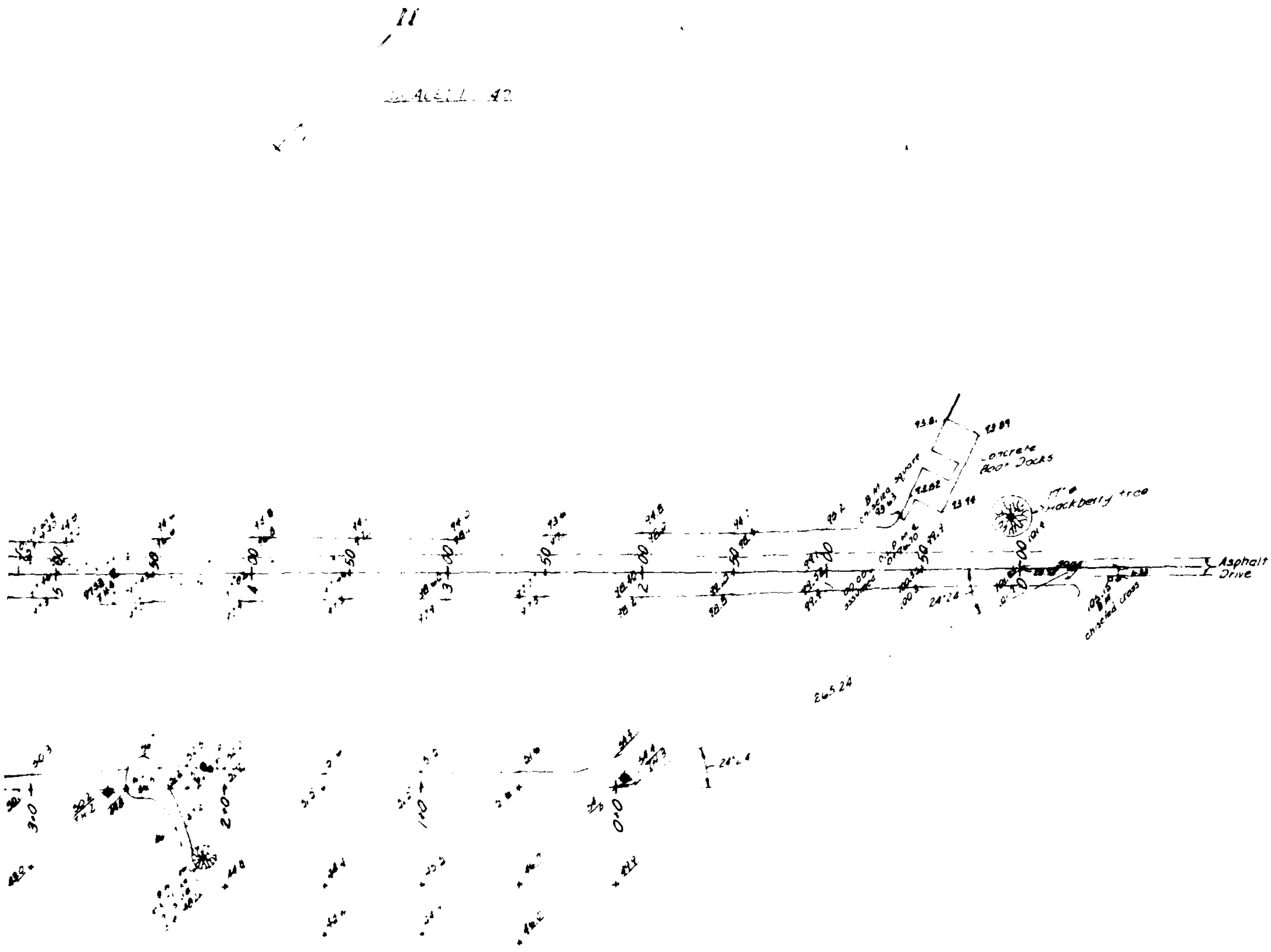


SUBDIVISION PLAT

PLATE 2

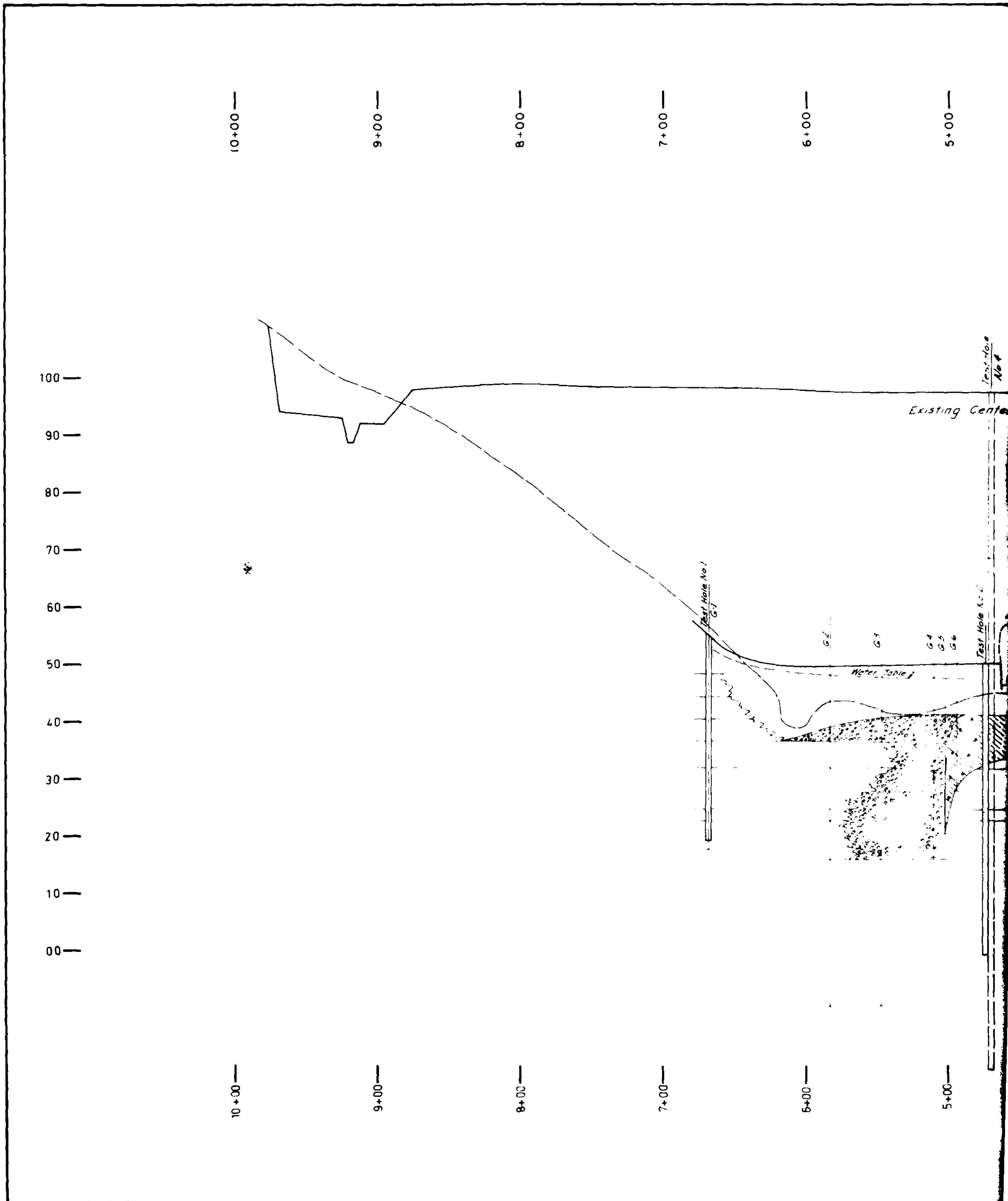


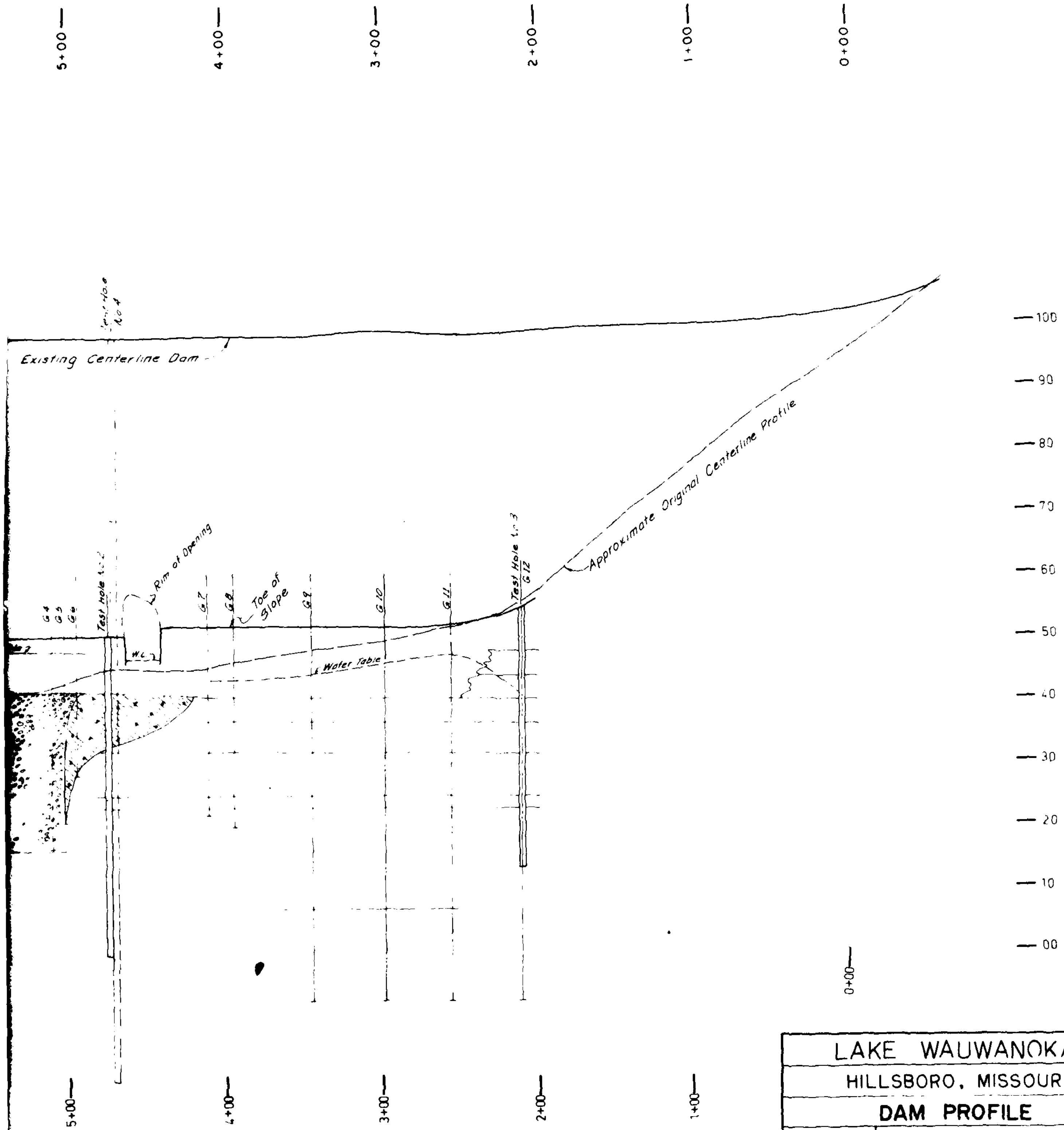
Concrete Boat Docks
93.09
93.07
Centerline of Dam



LAKE WAUWANOKA	
HILLSBORO, MISSOURI.	
DAM PLAN	
DESIGNED BY KRS DRAWN BY KRS CHECKED BY REVISIONS	BRUCKER & THACKER CONSULTING ENGINEERS RICHMOND HEIGHTS, MO.
	AUGUST 1967

PLATE 3 (SHEET 1 OF 5)


10+00
9+00
8+00
7+00
6+00
5+00
100
90
80
70
60
50
40
30
20
10
00
Test Hole No 4
Existing Cente
Test Hole No 1
G1
G3
Water Table
10+00
9+00
8+00
7+00
6+00
5+00



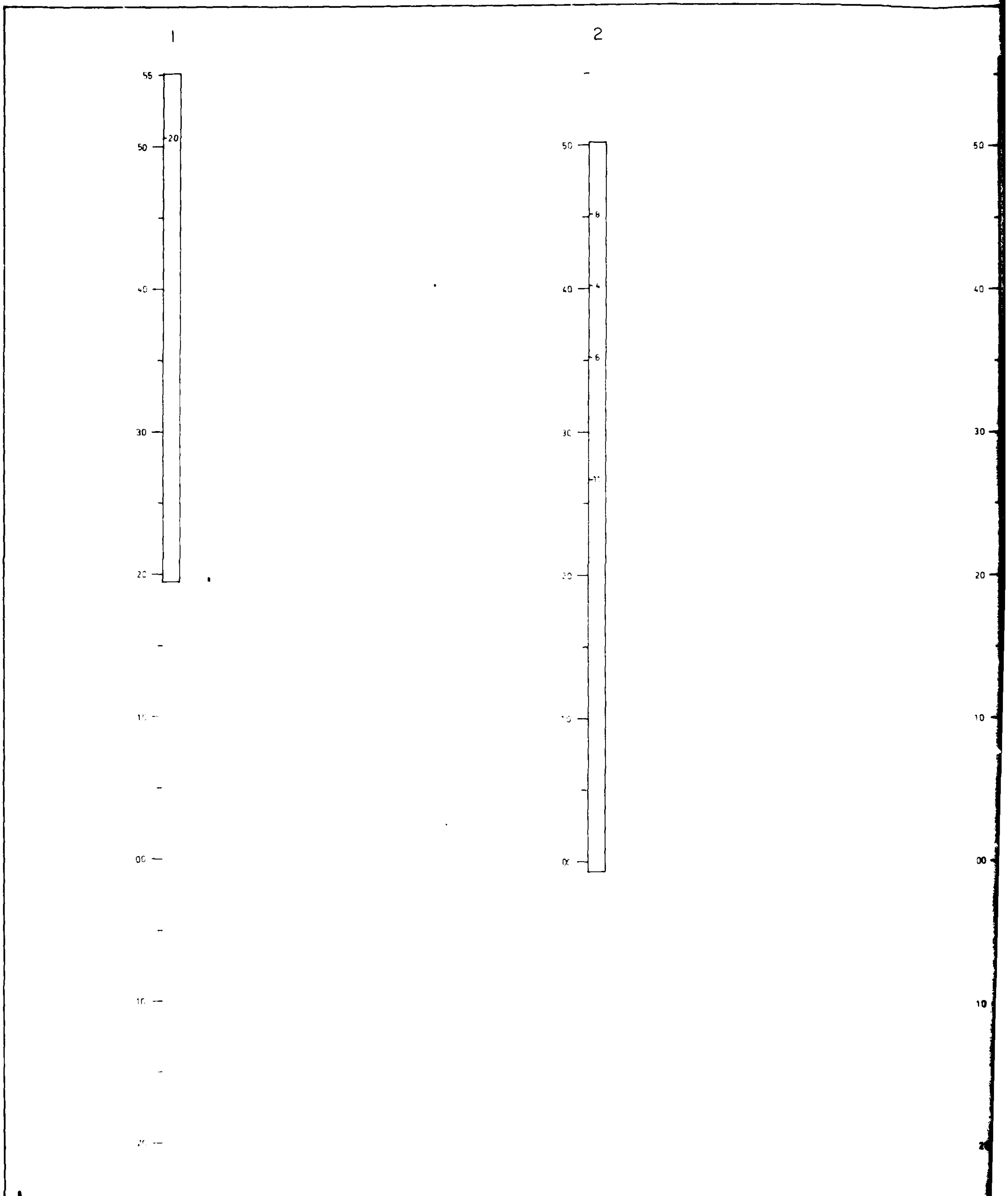
LAKE WAUWANOKA

HILLSBORO, MISSOURI

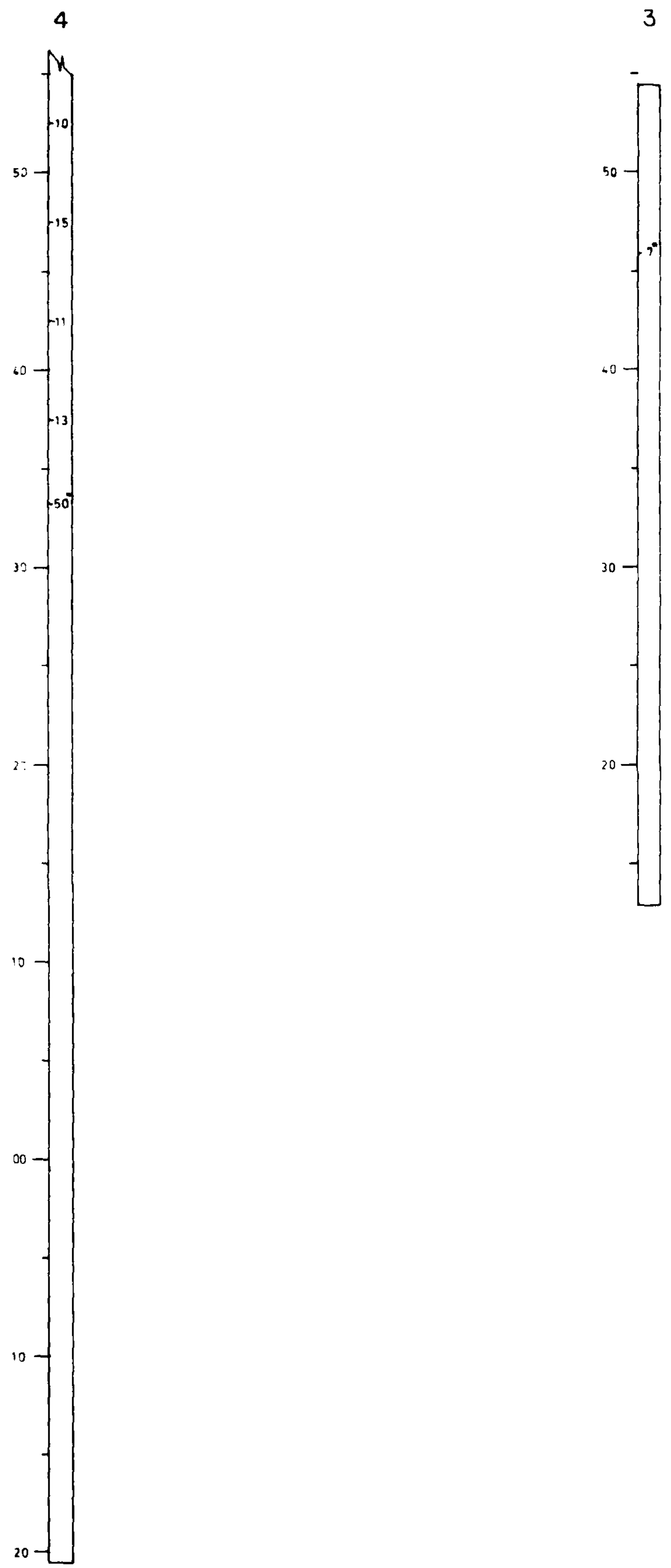
DAM PROFILE

DESIGNED BY KRS	BRUCKER & THACKER
DRAWN BY KRS	CONSULTING ENGINEERS
CHECKED BY	RICHMOND HEIGHTS, MO.
REVISIONS	AUGUST 1967

PLATE 4 (SHEET 2 OF 5)



1
2
55
50
40
30
20
10
00



LAKE WAUWANOKA

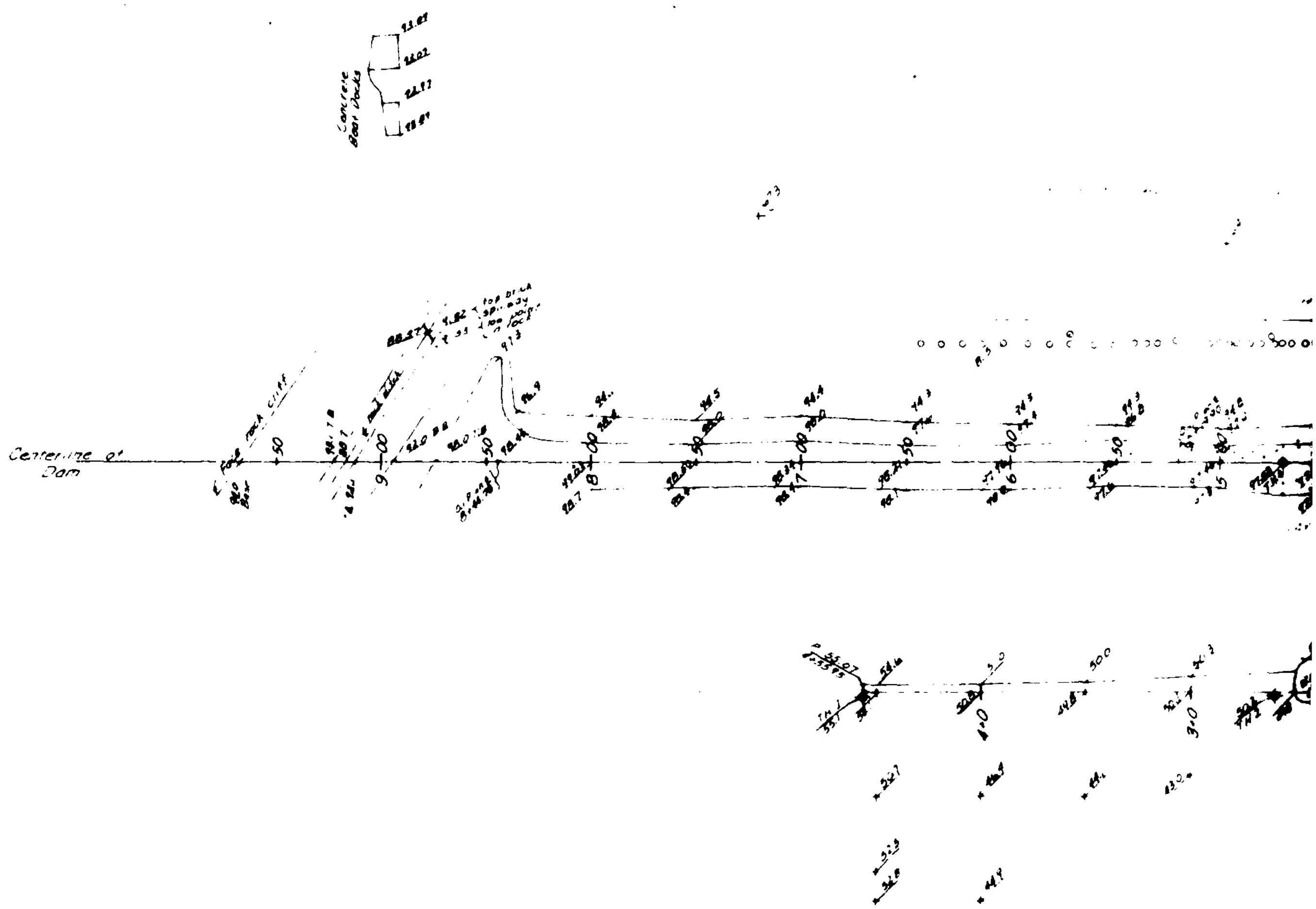
HILLSBORO, MISSOURI

BORING LOGS

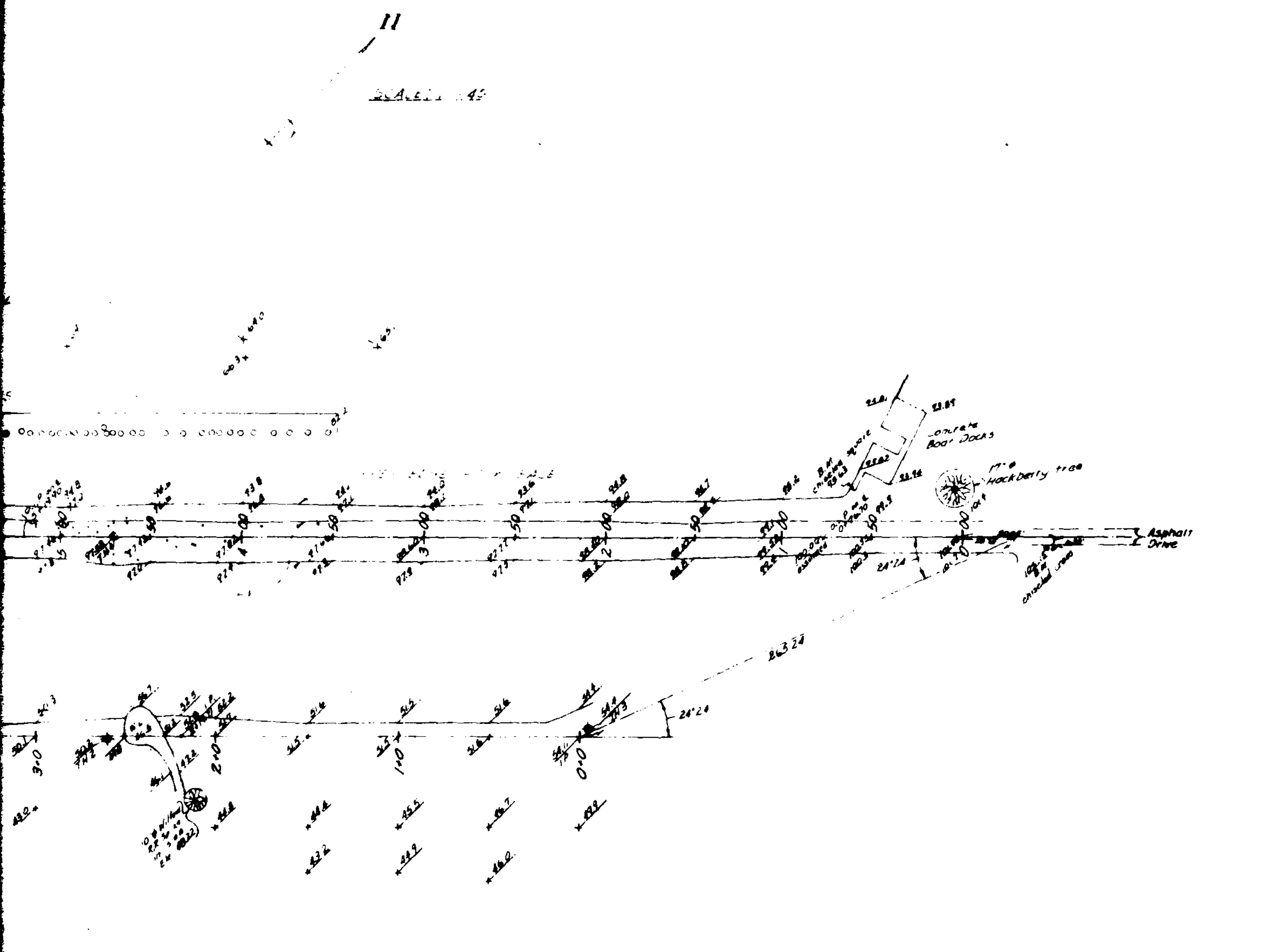
DESIGNED BY	BRUCKER & THACKER
DRAWN BY	CONSULTING ENGINEERS
CHECKED BY	RICHMOND HEIGHTS, MO.
REVISIONS	AUGUST, 1967

PLATE 5 (SHEET 3 OF 5)

12



Concrete Boat Docks
Centerline of Dam



LAKE WAUWANOKA

HILLSBORO, MISSOURI.

GROUT HOLE LOCATIONS

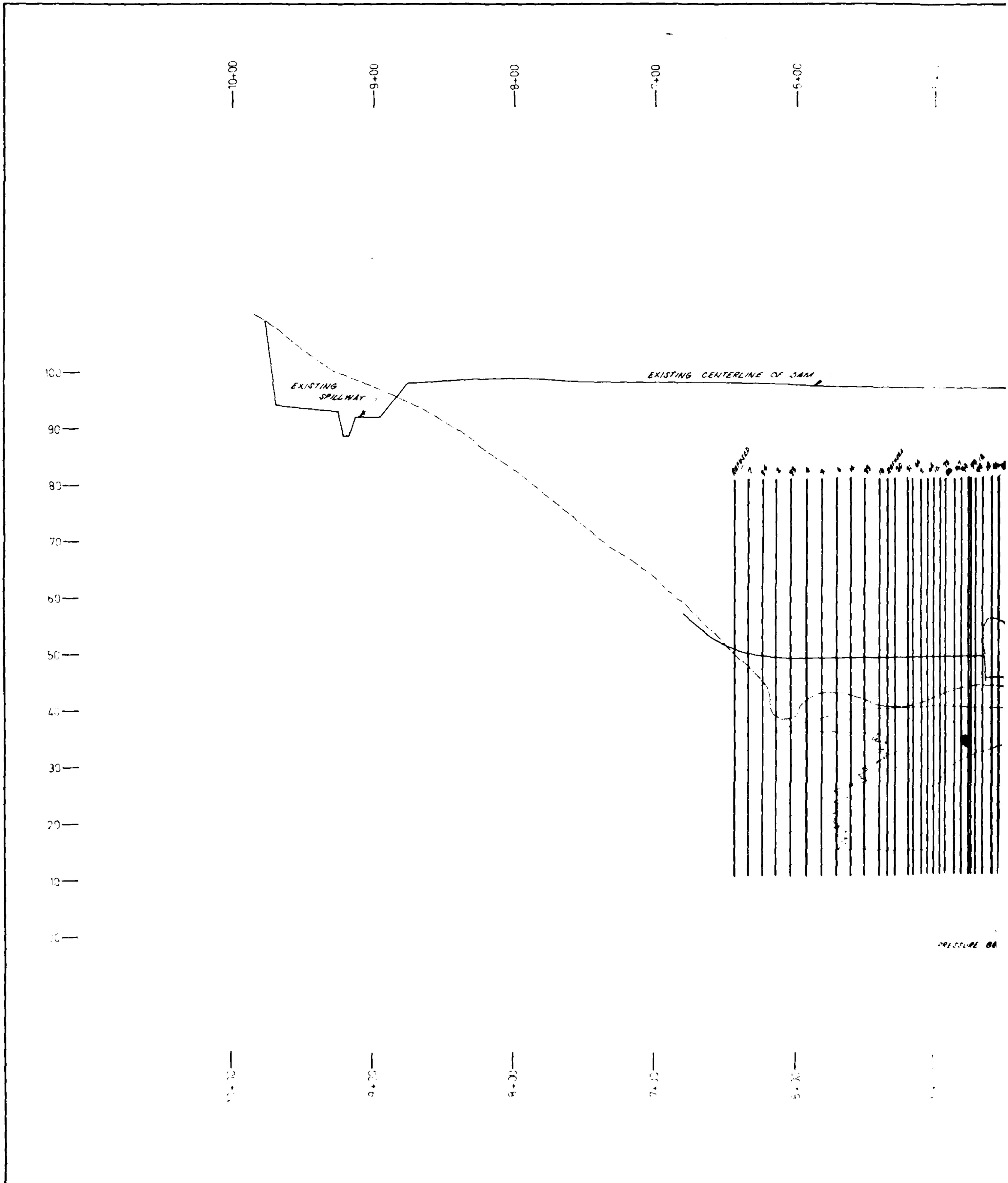
DESIGNED BY KRS
DRAWN BY KRS
CHECKED BY
REVISIONS

BRUCKER & THACKER
CONSULTING ENGINEERS
RICHMOND HEIGHTS, MO.

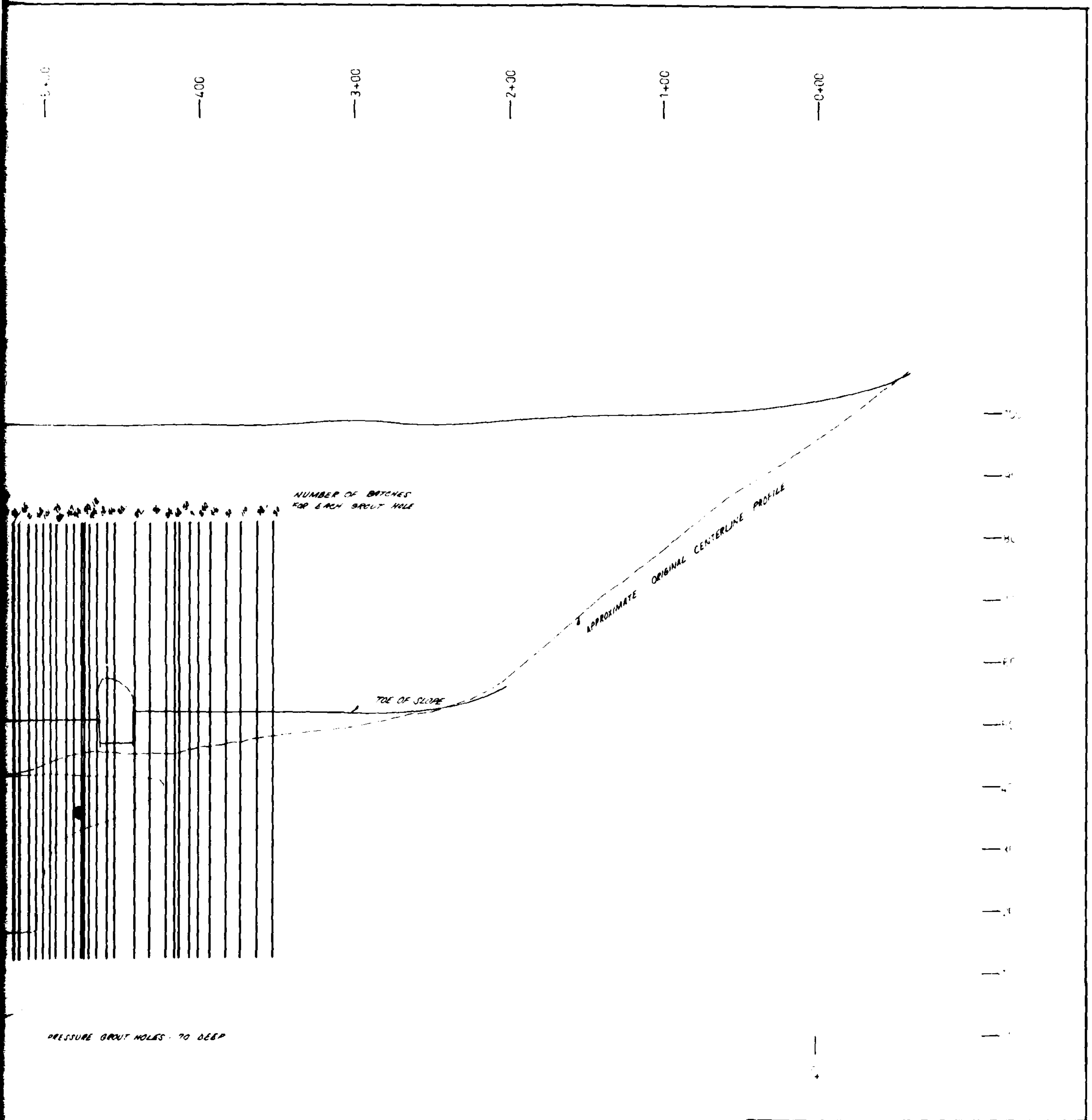
AUGUST 1967

PLATE 6 (SHEET 4 OF 5)

12



10+00
9+00
8+00
100
90
80
70
50
30
EXISTING SPILLWAY
EXISTING CENTERLINE OF DAM



LAKE WAUWANOKA

HILLSBORO MISSOURI

GROUT HOLE QUANTITIES

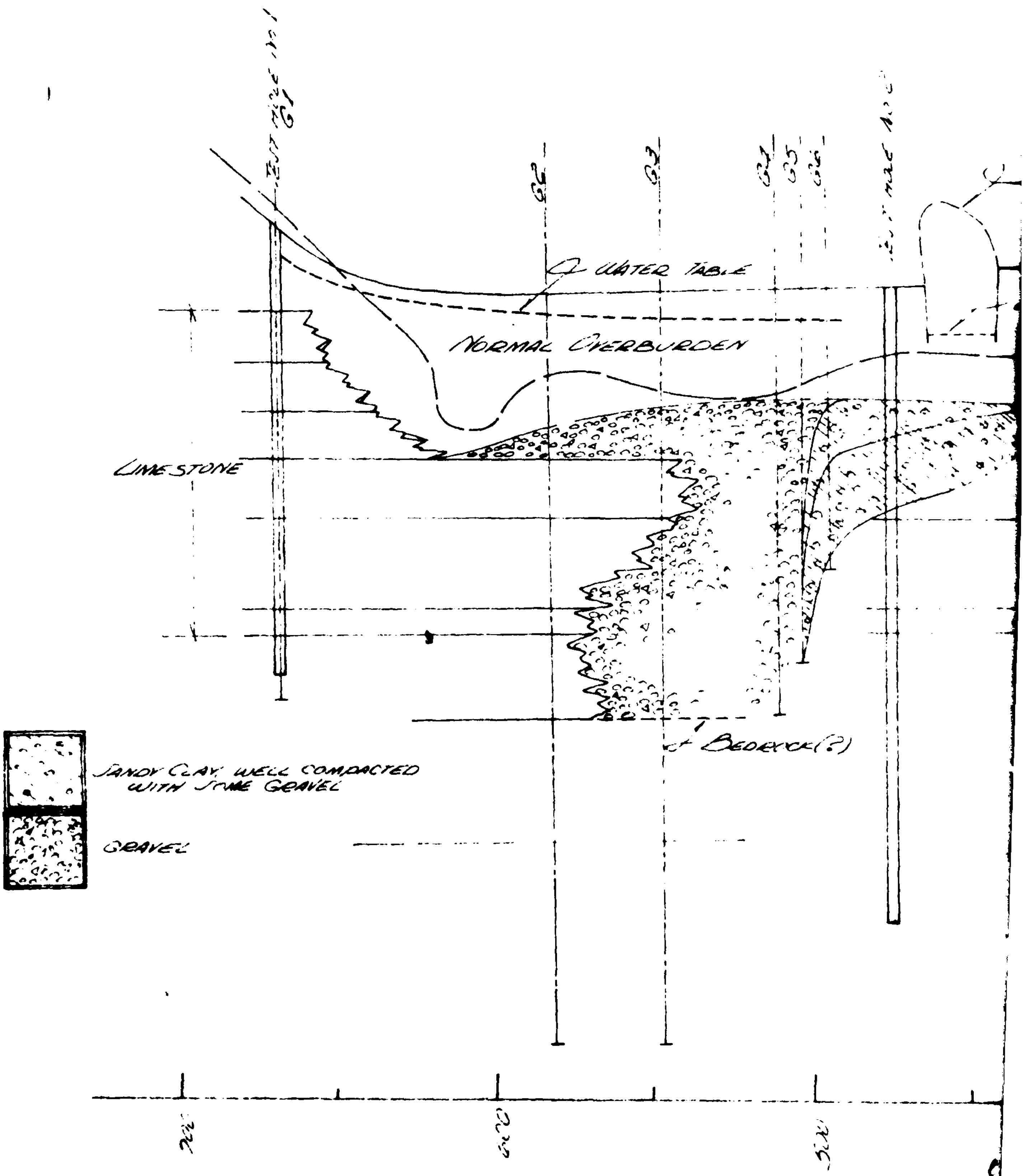
DESIGNED BY	
DRAWN BY	JM
CHECKED BY	
REVISIONS	

BRUCKER & THACKER
CONSULTING ENGINEERS
RICHMOND HEIGHTS, MO.

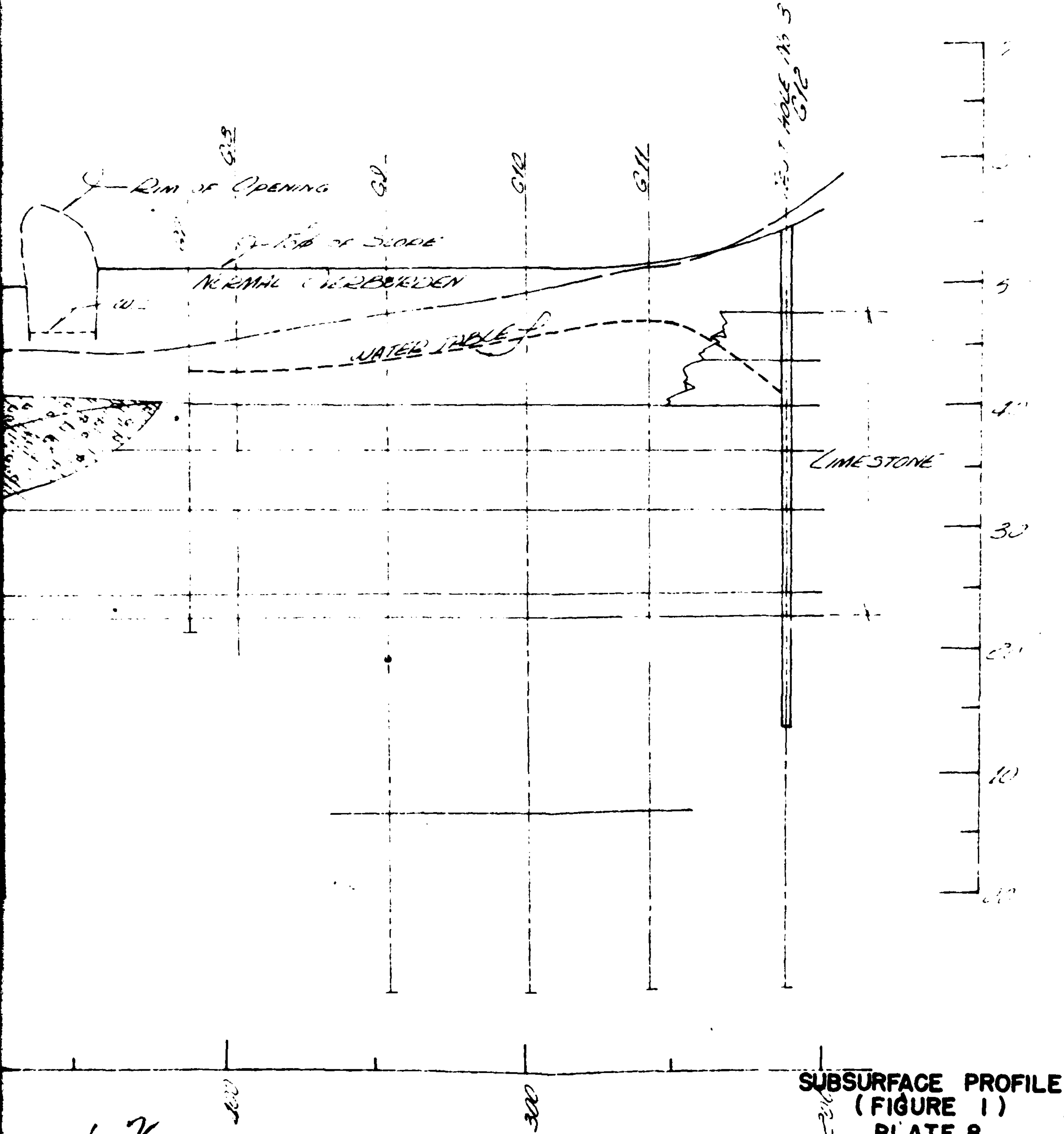
AUGUST 1967

PLATE 7 (SHEET 5 OF 5)

SUBSURFACE PROFILE OF LAKE G



LAKE WAUWANOKA DAM



SUBSURFACE PROFILE
(FIGURE 1)
PLATE 8

FIG. 1
SUBSURFACE PROFILE OF LAKE WAWANAKA
UPSTREAM SIDE

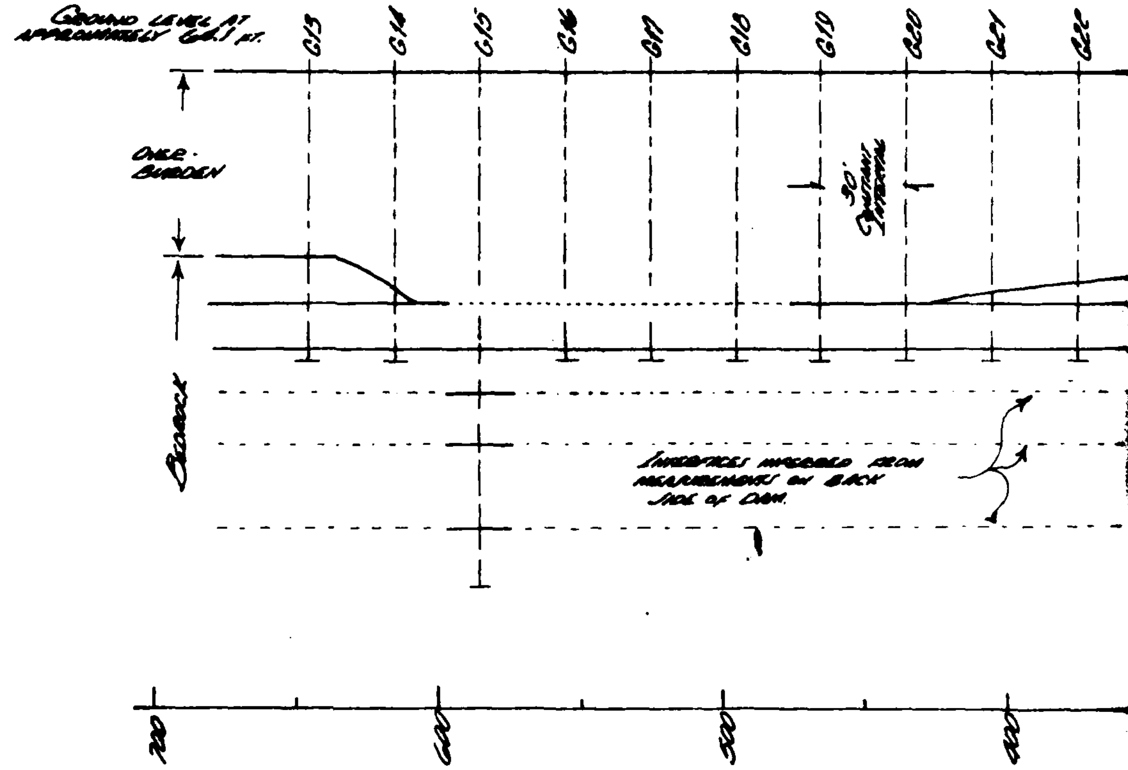
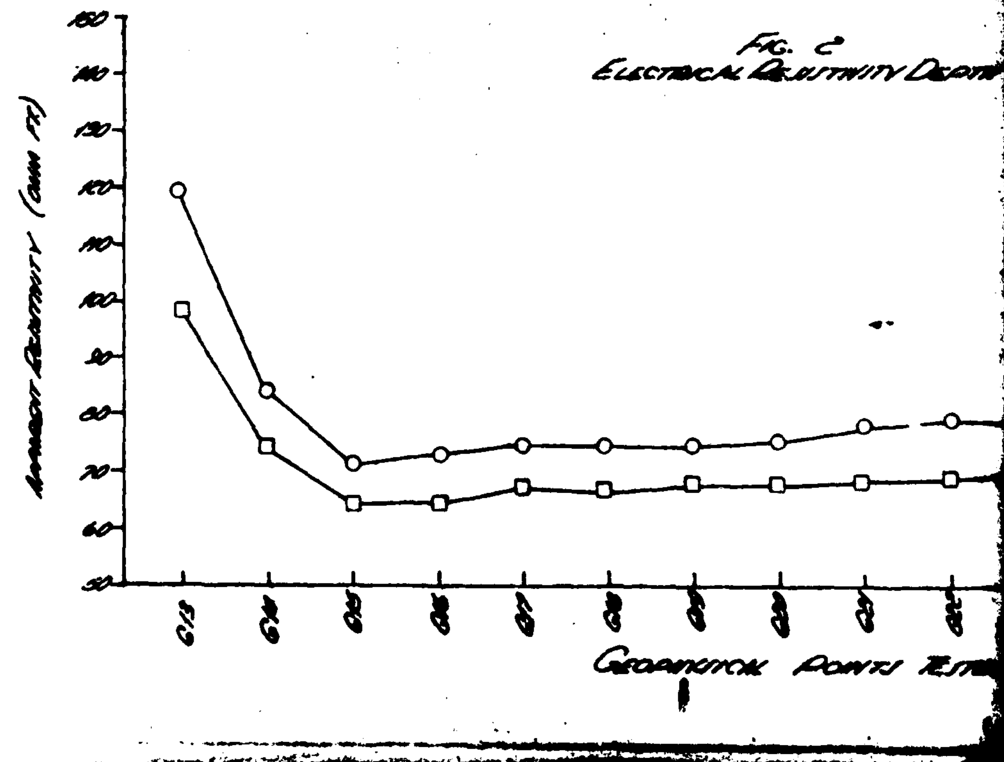
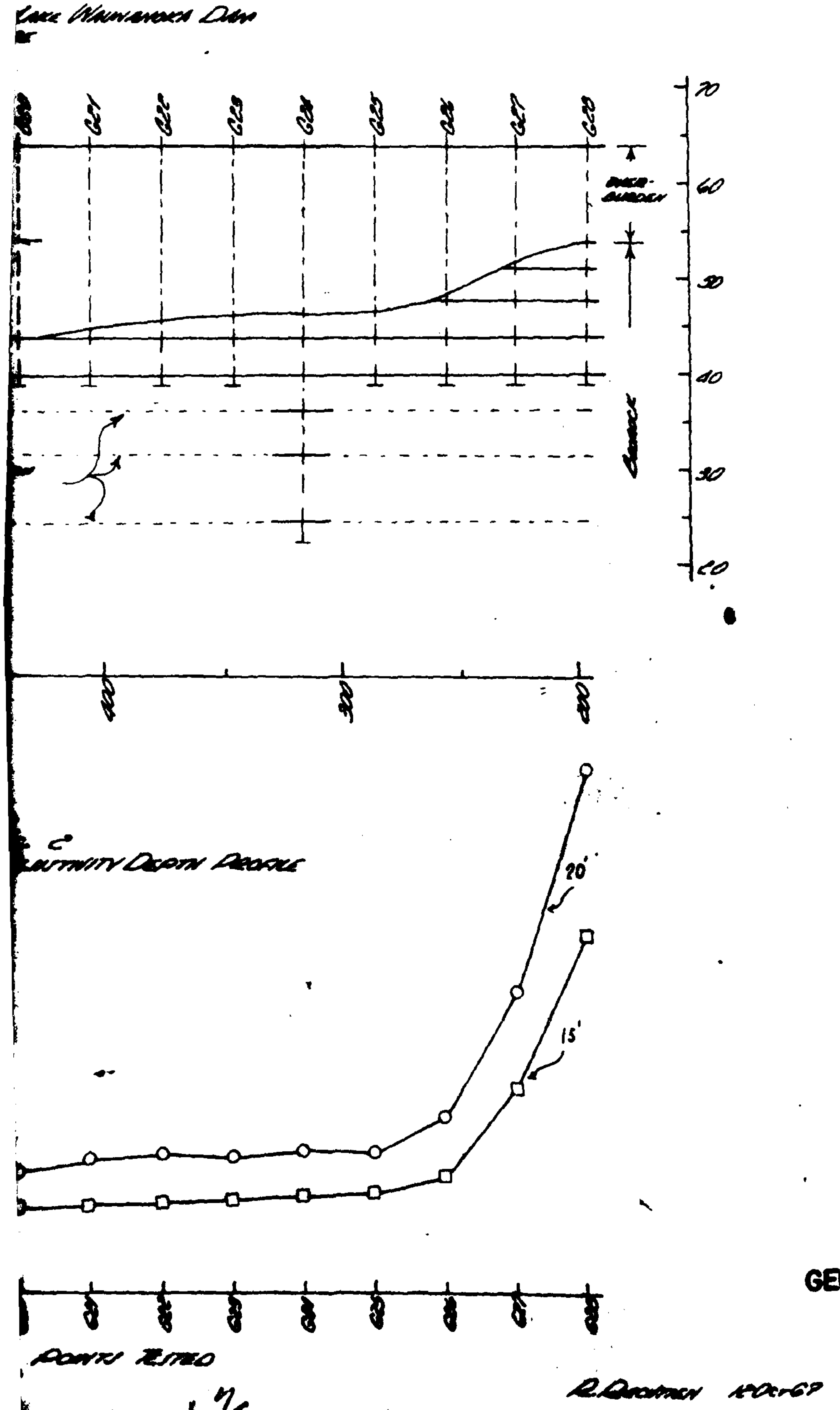


FIG. 2
ELECTRICAL RESISTIVITY DEPTH

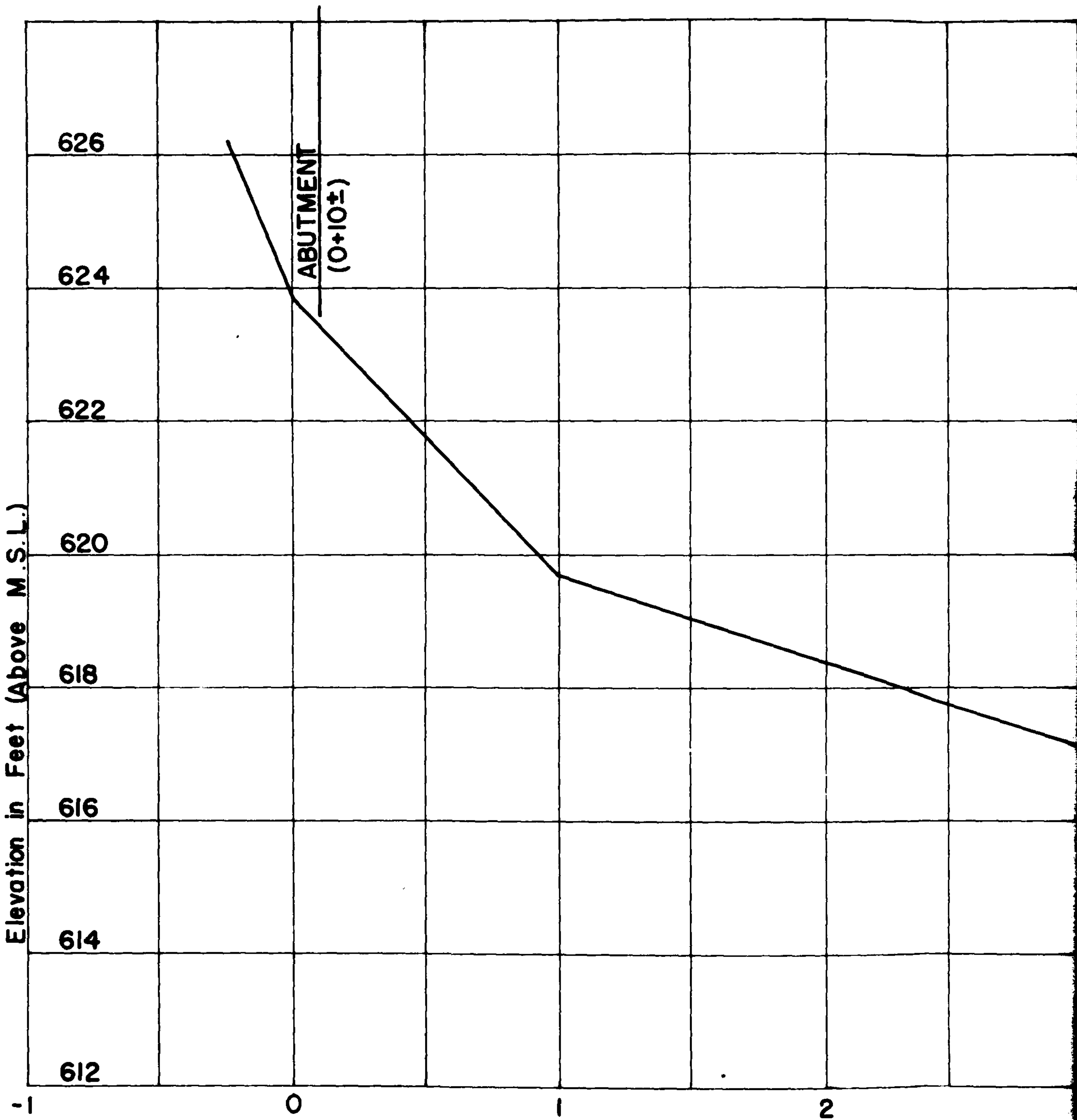




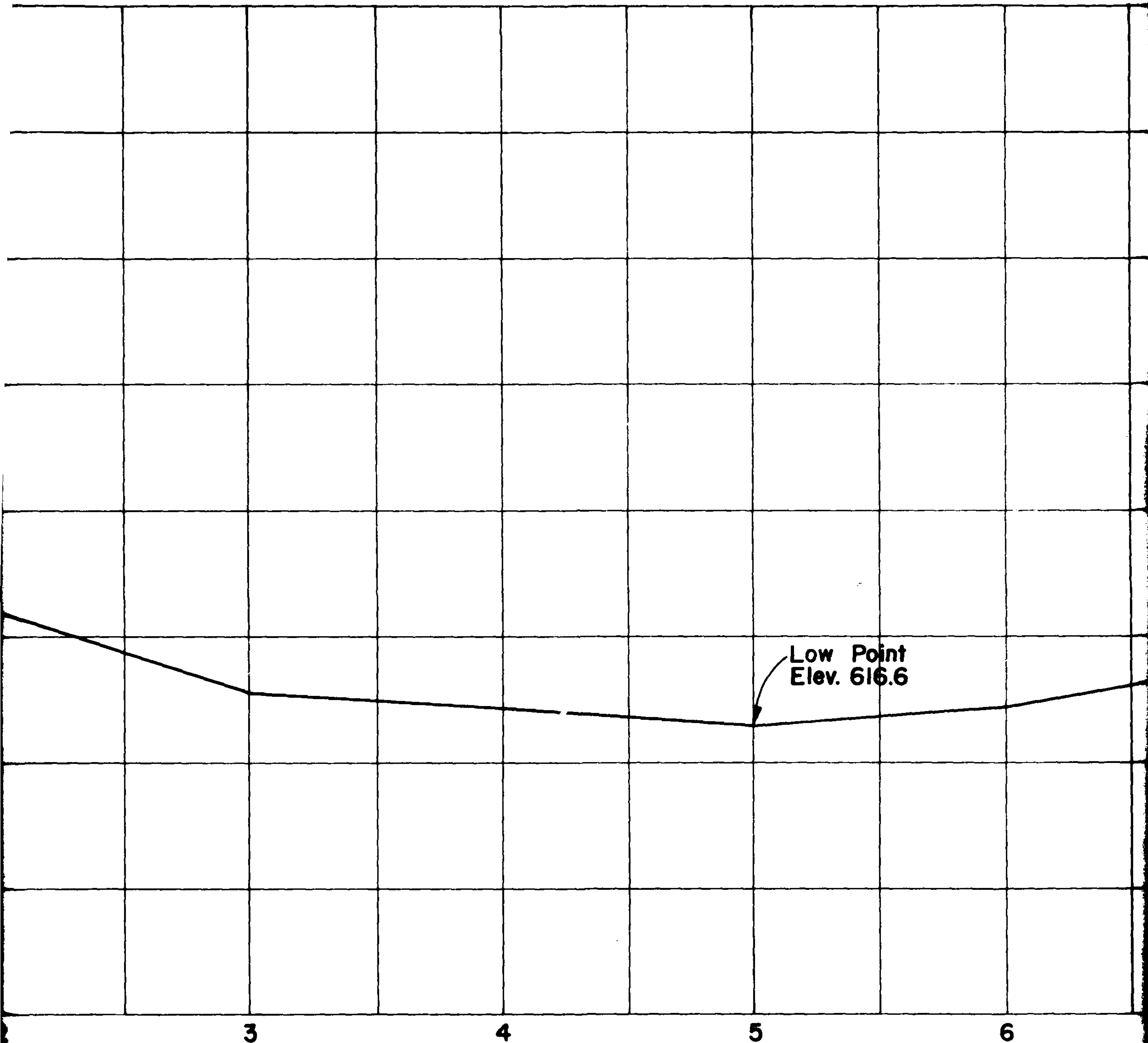
GEOPHYSICAL STUDY PROFILES

(FIGURES 1 & 2)

PLATE 9



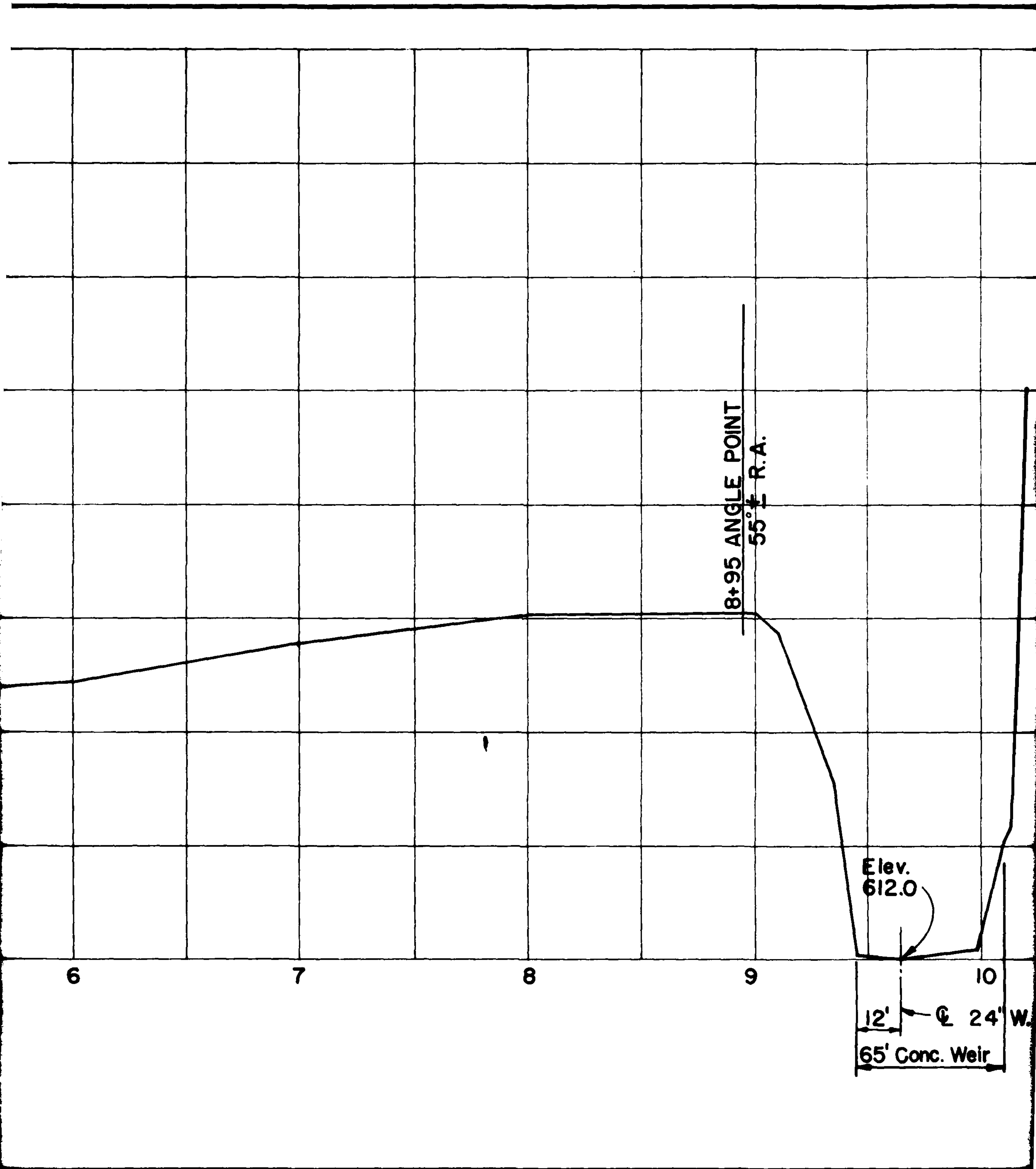
Elevation in Feet (Above M.S.L.)
626
624
622
620
618
616
614
612
ABUTMENT
(0+10±)
-1
0
1
2



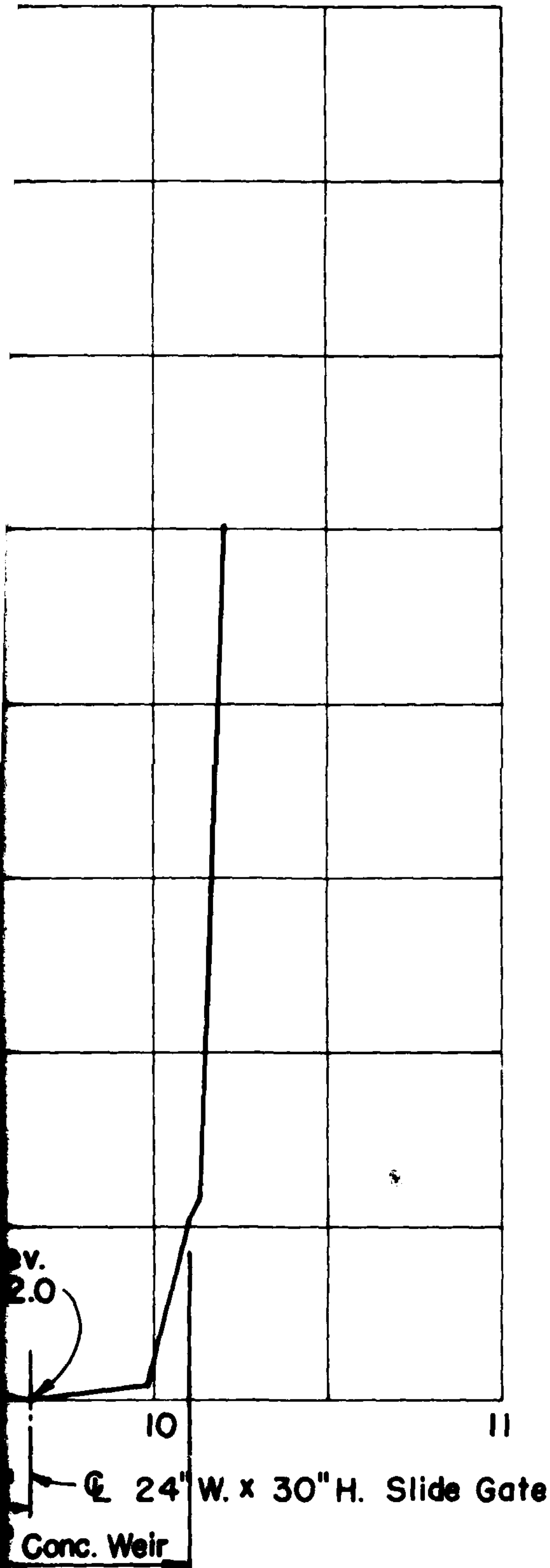
PROFILE DAM CREST

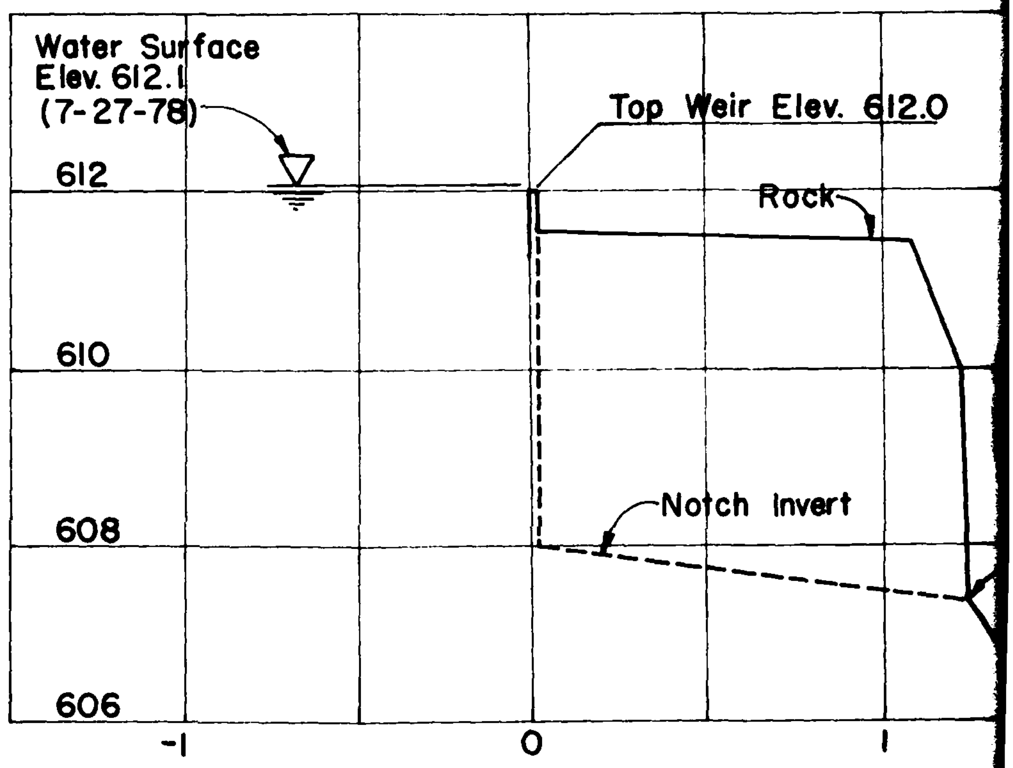
SCALES: 1"=2' V., 1"=50' H.

12



'3





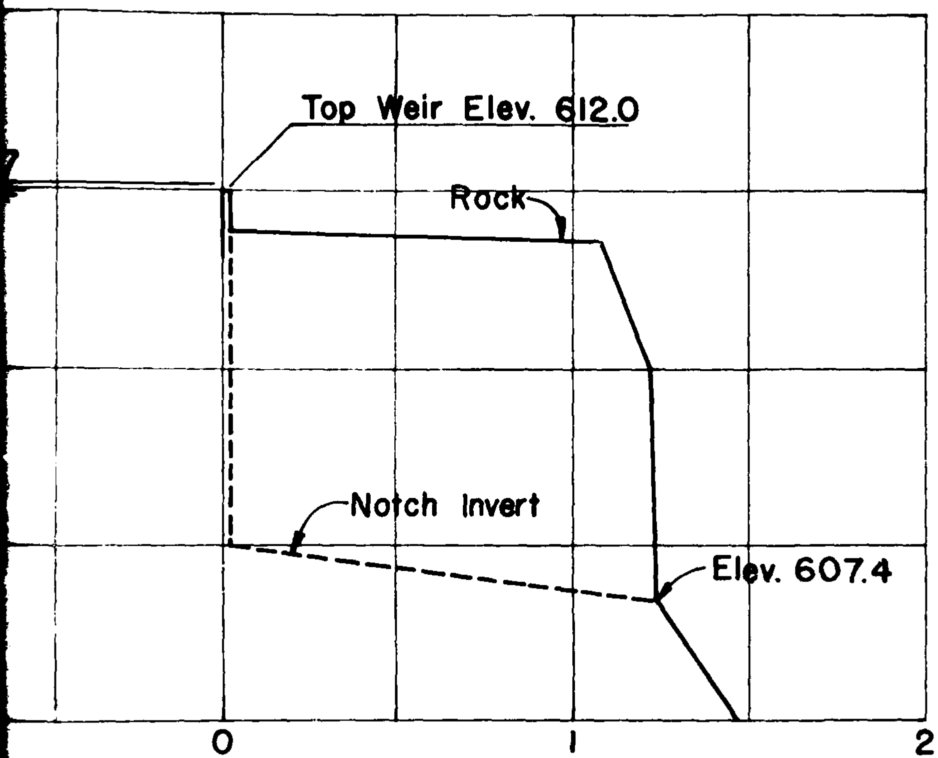
PROFILE SPILLWAY ℄

SCALES: 1"=2' V., 1"=50' H.

LAKE W

DAM & SPIL

Horner & Shifr



PROFILE SPILLWAY ℄

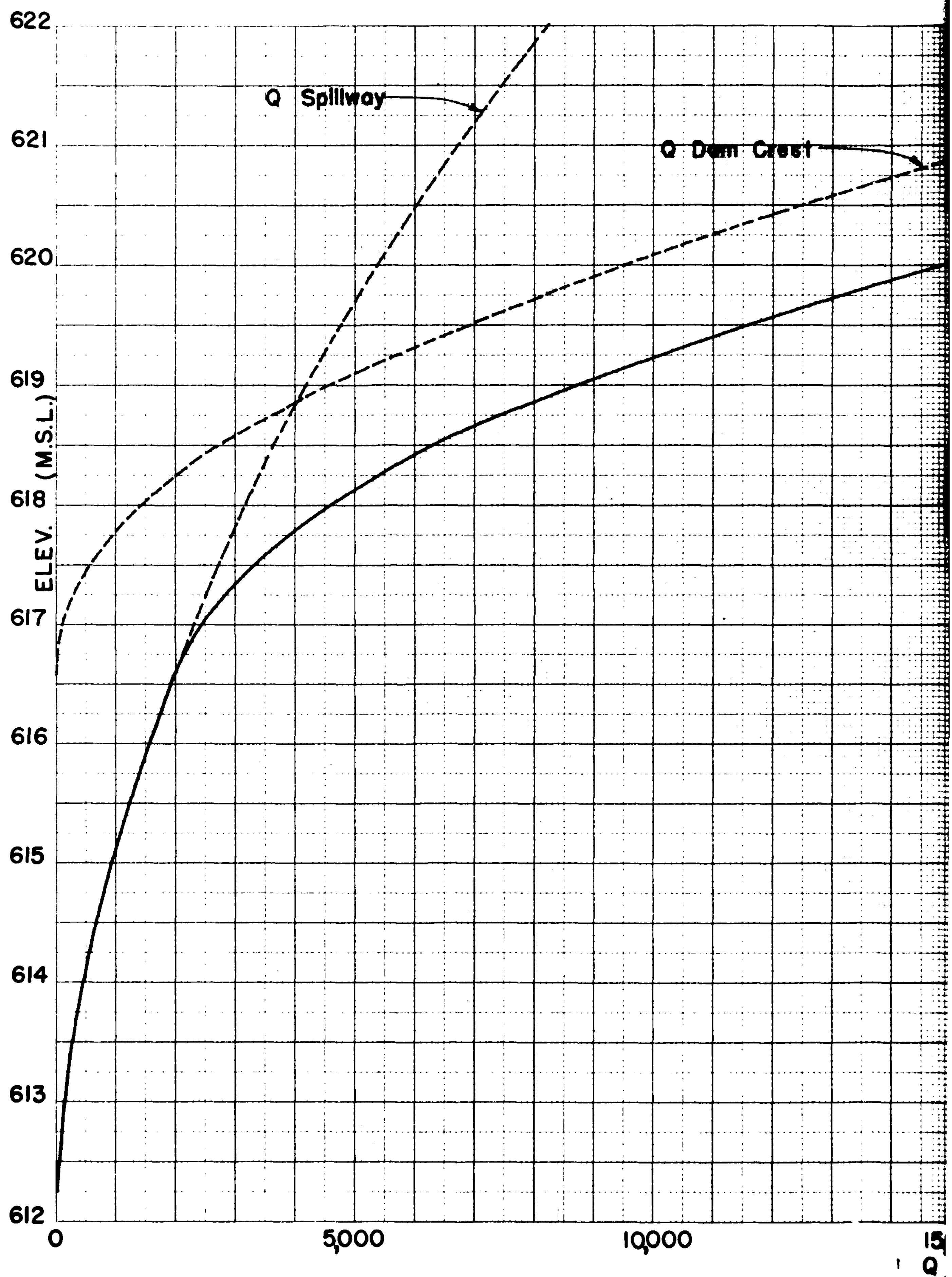
SCALES: 1"=2'V., 1"=50'H.

LAKE WAUWANOKA

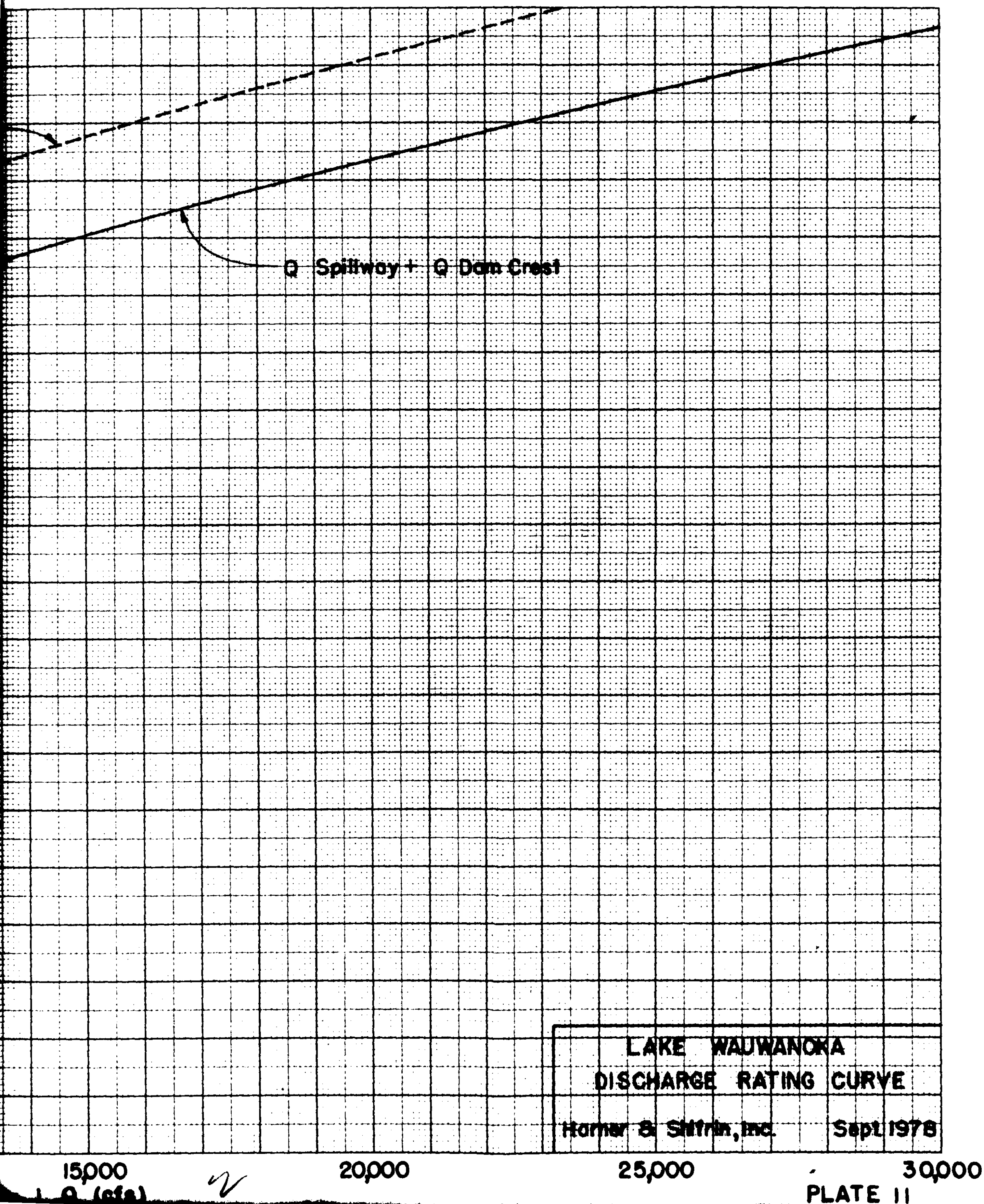
DAM & SPILLWAY PROFILES

Horner & Shifrin, Inc. Sept. 1978

PLATE 10

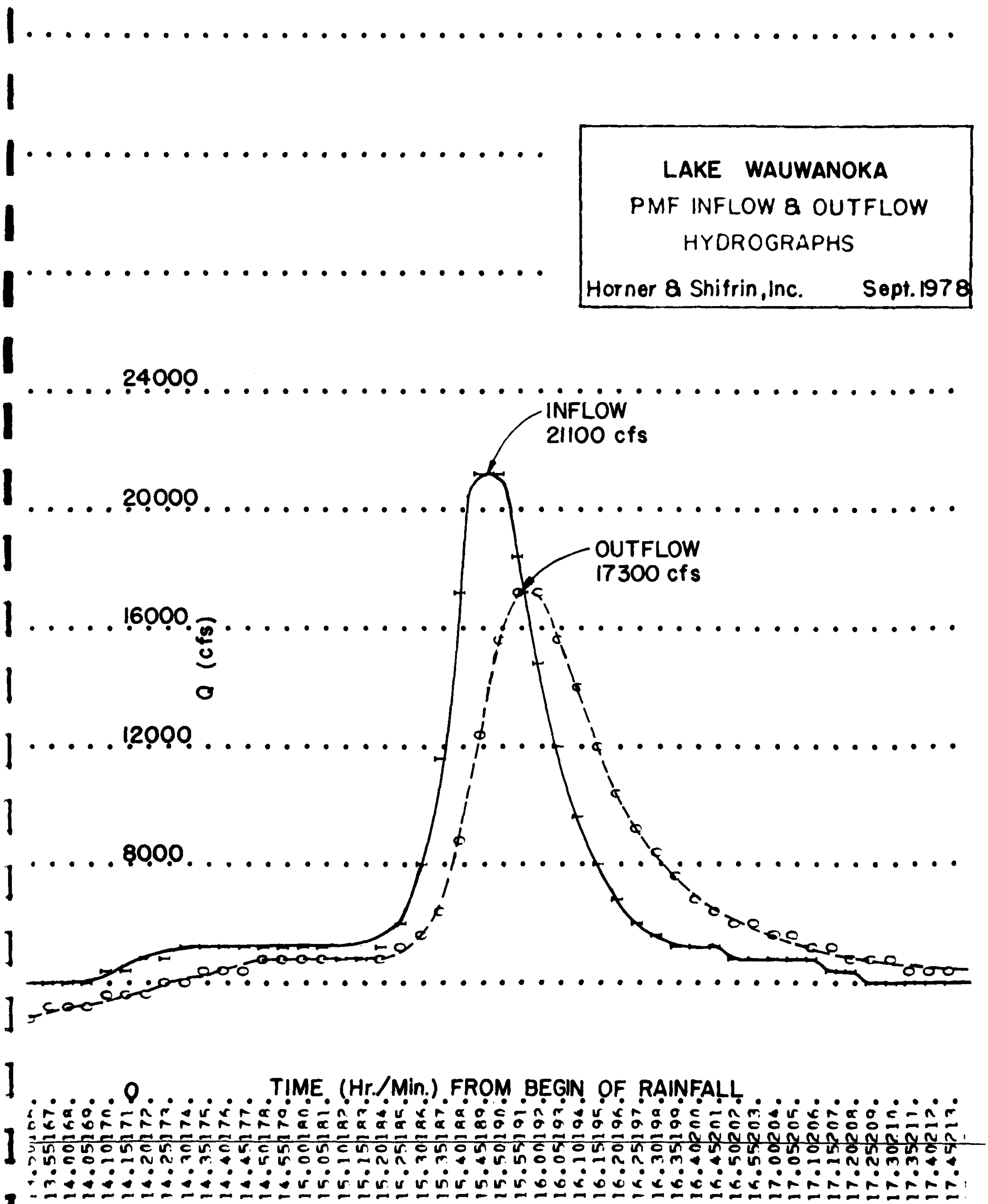


622
621
620
619
618
617
616
615
614
613
612
ELEV. (M.S.L.)
Q Spillway
Q Dam Crest
0
5,000
10,000
15
Q



Q Spillway + Q Dam Crest
LAKE WAUWANOKA
DISCHARGE RATING CURVE
Horner & Shifrin, Inc.
Sept. 1978
15,000
20,000
25,000
30,000
Q (cfs)

PLATE 11



LAKE WAUWANOKA
PMF INFLOW & OUTFLOW
HYDROGRAPHS
Horner & Shifrin, Inc.
Sept. 1978
24000
20000
16000
12000
8000
0
Q (cfs)
INFLOW
21100 cfs
OUTFLOW
17300 cfs
TIME (Hr./Min.) FROM BEGIN OF RAINFALL

PLATE 12

(Draft copy of report prepared by Brucker & Thacker, Consulting Engineers, about August 1967. Copy re-typed by Horner & Shifrin in order to improve legibility. Final report unavailable at this time.)

Investigation of Subsurface Conditions

LAKE WAUWANOKA
HILLSBORO, MISSOURI

Lake Wauwanoka approximately one and one-half miles east of Hillsboro in Jefferson County, Missouri, was built in the early 1940's. When full, the lake has an area of approximately 92 acres and a watershed of approximately 1350 acres giving a watershed to lake ratio of approximately 14-1/2:1. The dam was built across a branch of Dry Creek by Mr. Schielly of Kansas City, Missouri, who was referred to in older correspondence as "the engineer who laid out the original project at our lake." Most of the information concerning the preliminary planning, construction details and plans has been furnished by older residents at Lake Wauwanoka.

This office was retained by Lake Wauwanoka, Inc., to investigate the source of leakage beneath the dam at Lake Wauwanoka and to make recommendations for corrective work. Verbal discussions held with various lot owners and others associated with Lake Wauwanoka at various meetings indicated a divergence of views as to the original construction and a _________ of XXXXXXXXXXX written records. The earliest information of a written nature was a letter X of June 1952 written by Mr. Dierkes, a member of the board of directors of Lake Wauwanoka, of which the following is a partial quote outlining the beginnings of the dam construction as counted by Mr. Dierkes from information he was able gather at that time.

"When the dam was built, the contractor cleared all earth and gravel from the site to solid limestone even to the extent that the surface was swept clean with brooms. However, on the south side about 250 feet from the spillway they encountered a

of sandstone of brownish color about 3 feet deep and 40 to 50 feet in length which he XXX stated was removed and a facing of rock and concrete was placed on both sides. The key and core of the dam were then built of good, clean clay and the dam completed having a 3 to 1 slope on the water side and a 2 to 1 XX slope on the lower side. Since the leak is in evidence in that portion of the structure he feels that possibly a leak may have developed through this point."

The only other written information and records that could be obtained were a cross section along the centerline of the dam which was furnished by Barney Schubel and several logs furnished by Walter Freuler covering generalized logs of some of the holes which were pumped with asphalt during the period of June 12, 1967 through July 26.

Investigative Program

Because of lack of information with regard to investigative work construction and remedial work down the dam this office recommended that topography be prepared of the dam area before investigative work was started so as to have some base map on which to plot and evaluate information. This work was completed in the vicinity of the dam.

Core Drilling

Four test holes were drilled with a truck-mounted continuous flight auger rig compable of coring rock. The first three test holes were drilled on the downstream face of the dam at the locations shown on Figure 1. These test holes were drilled beneath the ground surface until approximately 30 feet of good rock had been encountered. In addition, a fourth test hole was

drilled along the centerline of the dam through the core and extended a total depth of approximately 110 to 118 feet. This test hole was drilled in an area XXX through which previous information had indicated a likelihood of water entering the dam and also to get information on the core of the dam itself. Although these test holes were widely scattered, they were to serve as controls for geophysical work which was also planned. Logs of the test holes showing the nature and thicknesses of the various rock types encountered and the percent of core recovery are shown on Figure _____. In addition, photographs of the cores are shown on Figures 3-1 through 3-4.

3. Geophysical Investigation

Geophysical work using electrical resistivity was performed along the downstream face of the dam XX in a line with the three test holes previously drilled. Vertical profiles were run at 12 locations along the downstream face of the dam. These locations are numbered G-1 through G-12 and were carried where possible to depths of 60 feet. In addition, two horizontal resistivity profiles were taken along the base of the dam at intervals of 5 feet. The depths of these horizontal profiles were 15 and 20 feet respectively. These profiles were employed to determine any horizontal discontinuity of the rock structure at these depths. In general, the rock to the north of the leak in the dam consisted of relatively sound bedrock XXXXX consisting of several uniform layers of limestone overlain by approximately 12 feet of overburden. Similar information and rock structures were found on the south edge of the dam between geophysical stations G-1 through G-3. However, between G-3 and the leak the type of data obtained was very eratic suggesting a highly permeable material probably gravel. The interpretation

from the geophysical data is shown on Figure _____. Additional geophysical investigative work was attempted approximately halfway up of the downstream face of the dam but the presence of asphalt completed prevented any readings based upon electrical resistivity. Because of the riprap on the upstream face of the dam, no electrical resistivity work was attempted in this area nor on top of the dam XXX since the depths were becoming increasingly large.

Leakage Beneath the Dam

Since our initial observation of the dam on July 31, 1967, it appears that the rate of loss of water has been roughly constant. It has been estimated to be on the nature of 5 to 5 million gallons a day although to our knowledge no actual measurements of the rate have made. A minor amount of sluffing of the embankment has occurred on the downstream source of the dam adjacent to the leak, exposing thin layers of sand and gravel in the embankment at this point. The diameter of the hole is approximately 12 to 15 feet. Water appears to be rising vertically through the gravel which is in a quick condition and frequently the force of the water is sufficient to break the surface of the pool. The gravel at the points of water escape is at a constant state of agitation and asphalt can be observed escaping in the stream which flows away from the dam. At all times of our observation, the water is relatively clean indicating little or no loss of soil and thus implying that the water is being XX filtered through gravel fracture or porous limestone and not causing or allowing any loss of soil.

Investigation

The investigative work performed under supervision of this office consisted primarily of core drilling and geophysical work.

Three test holes were drilled along the downstream slope of the dam. The locations of these test holes are shown on Figure 1. Test Holes 1 and 3 were carried approximately 30 feet into solid rock where core recovery was good to excellent. These cores indicated that there was little likelihood of loss of water at either of these two points. Test Hole 2 was drilled immediately south of the leak in the downstream slope of the dam. Rock was encountered at approximately 20 feet below the surface and the cores show a high degree of fractured limestone.

A fourth test hole was drilled from the crest of the dam. This test hole was drilled in an area where considerable holes had been previously drilled and into which asphalt had been pumped. The purpose of drilling this hole was to verify the nature and consistency of the material in the dam and core and also to check to see whether or not the asphalt grouting had been effective in this area as well as to check for similarity in rock conditions. The rock cores encountered in this test hole were comparable to those in Test Hole 2 which consisted of fractured limestone. Some asphalt was also encountered of a relatively minor nature as shown on the logs.

The following of avenues of corrective work were explored standpipe over the leak.

STANDPIPE OVER THE LEAK

This idea was initially suggested when information available to us indicated that rock was probably 8 to 11 feet beneath the ground surface where the leak is occurring and

because we had heard that higher leakage from this point had always been fairly well concentrated in a small area. The use of the standpipe over the leak had been suggested in conjunction with pressure grouting of the voids through which the water was percolating. This idea sound from an engineering standpoint appeared to have too many difficulties associated with it. These consisted of the following. Test Hole 2 indicated that rock good sound rock was approximately 23 feet below the surface below this point. One of the contractors who previously worked in this area indicated the large boulders the size of half a cubic yard or more had been pushed into the area where the leak is occurring and had sunk out of site due to the "quick" condition. As a result, excavation down to the rock would have been far in excess of what had originally been anticipated and the possibility of sinking a casson-type structure into the gravel which is in a "quick" condition indicated the likelihood of poor or no success due to the presence of large boulders on top of the rock on which the casson structure might easily become hung and a good seal between the casson and the rock being prevented.

A MEMBRANE OVER THE LAKE BOTTOM:

Although this was never seriously considered by this office due to the relatively high cost, approximately $1.00 per square yard for material alone, this possibility was rejected on two accounts: (1) information available indicated that during the excavation for the lake the soil cover which was originally thin in its natural condition had been removed down to gravel or shattered rock.

PRESSURE GROUTING:

Although the pressure grouting was part of the original idea in conjunction with the standpipe, the primary purpose of the standpipe was to prevent the escape of the water and also to slow the water down sufficiently so that grouting could be more easily undertaken.

[illegible]

TEST HOLE 2:

0-6'	Brown, moist, clayey [illegible] with small gravel and small rock fragments.
6'-12'6"	Brown with trace of grey, moist, silty clay; trace of small gravel, and trace of fine sand.
12'6"-18'6"	Brown clay with fine sand and small gravel.
18'6"-19'	Rock or Boulder.
18'6"-23'6"	Cored 5', Recovered 48"; Brown limestone with Intermittent seams.
22'6"-24'	Gravel and decomposed limestone; trace of asphalt tar in tip of spoon.
24'-26'	Decomposed limestone; asphalt tar coming up through wash.
26'-31'	Cored 5', Recovered [illegible]", white limestone.
31'-36'	Cored 5', Recovered 57", white and grey limestone with spots of chert rock.
36'-40'9"	Cored 4'9", Recovered [illegible], grey limestone with spots of quartz, spots of sandstone, some small seams blue shale.
40'9"-45'9"	Cored 5', Recovered [illegible], brown, fragmented limestone with spots of chert
45'9"-47'3"	Cored 1'6", Recovered [illegible], brown, fragmented limestone [illegible]
47'3"-50'10"	Cored 3'7", Recovered [illegible], brown, fragmented limestone with spots of chert rock.

LAKE WAG[illegible]

TEST HOLE 3:

0-4'3"	Boulders and [illegible] clay.
4'3"-6'9"	Rock/or boulder.
6'9"-8'6"	Cored 1'9" [illegible], [illegible] brown sandstone and hard blue shale. [illegible] of brown sandstone and 11' of blue shale with sandstone layers.
8'6"-8'9"	Brown, moist clay [illegible] decomposed [illegible]stone.
8'9"-11'9"	Rock or boulder; Cored 3', Recovered 31", Brown and grey limestone with small layers of sandstone with intermittent seams of decomposed sandstone.
11'9"-16'9"	Cored 5', Recovered [illegible], grey limestone.
16'9"-21'9"	Cored 5', Recovered 59", grey limestone with spots of brown.
21'9"-26'9"	Cored 5', Recovered [illegible] brown and grey limestone.
[illegible]'9"-31'5"	Cored 4'8", Recovered [illegible], Grey limestone.
31'5"-36'5"	Cored 5', Recovered [illegible], grey limestone.
[illegible]'5"-41'5"	Cored 5', Recovered [illegible], grey limestone.
	Water level 13'[illegible]" at completion.

Chart 2-9

26 August, 1967

TO: Brucker & Thacker
1045 S. Brentwood Blvd.
Richmond Heights 17, Mo.

FROM: Dr. Richard D. Rechtien
Geophysical Consultant
4 Crestview Ave.
Rolla, Missouri

SUBJECT: Report of the Geophysical Investigation of the Lake Wauwanoka Dam, Hillsboro, Missouri

INVESTIGATION OBJECTIVE:

The purpose of the investigation was to determine the condition of the subsurface material underlying the Lake Wauwanoka Dam in an effort to detect the point, or points, of water leakage beneath the dam.

FIELD INVESTIGATION:

The electrical resistivity method was employed to investigate the subsurface. Field work was conducted on the 21, 22 and 25th of August, 1967.

Vertical profile data was taken at twelve locations along the base of the dam. These locations are shown in Figure 1. Where possible, data was taken to depths of 60 feet.

Two horizontal resistivity profiles were taken along the base of the dam at intervals of 5 feet. The depths of investigation were 15 and 20 feet respectively. These profiles were taken to determine any horizontal discontinuity in the rock structure at these depths.

RESULTS:

The results of this investigation are summarized in Figure 1. This figure is a direct overlay of the dam profile figure supplied by G. Brucker. The actual resistivity data is not presented as it would be quite meaningless to the untrained eye. This data, however, is available upon request.

The resistivity data was correlated with core data taken in three

drill holes. The location of these holes are also shown in the figure.

The resistivity data taken at test points G-7 through G-12 was uniform and correlation of the substrata from one test point to another was relatively straightforward. The data taken in this region showed a sequence of level, uniform layers of limestone overlain by 12 feet of overburden. The electrical resistivity data of G-7 through G-11 indicated that the limestone layers in this region were slightly more weathered than in the vicinity of G-12 and consequently had a slightly higher permeability than normal. The electrical resistivity of the overburden was indicative of a compacted clay with some sand and gravel.

The data from G-1 and G-2 and the shallow data from G-3 correlated well with the core data and with the geophysical data from G-7 through G-12. The 12 foot depth interface, however, was not detected at test points G-2 through G-6. Bedrock was detected at G-2 and G-3 at a depth of 13.5 feet relative to ground level at these points.

The measurements taken at test points G-5 and G-6 were restricted to depths of 35 and 25 feet respectively due to requirements of electrode separation and the proximity of the water outflow at the base of the dam. The data taken at these test points and the data from G-4 and the deeper data from G-3 showed quite different results than from the other test points and did not correlate with the other geophysical data. In reference to the figure, the data from G-5 and G-6 and from the horizontal profiling showed a discontinuity in the shallow limestone layers and bedrock was found at a depth of 30 and 20 feet respectively. The bedrock here was overlain by two distinguishable layers of a well compacted clay-type material.

The data at G-4 was taken to a depth of 34 feet. The data here was very erratic and was indicative of a clay-type material down to a depth of 15 feet and underlain by gravel. Bedrock for this test point was not clearly discernible, but indications were that it may be located between 33-35 feet in depth.

The deeper data from G-3 complimented that from G-4, indicating the presence of a gravel at depths between 21 and 34 feet, below which it appeared that solid rock again existed.

CONCLUSION:

In view of the electrical resistivity and core data, a subsurface condition summarized by Figure 1 is concluded. These data strongly suggest the presence of a highly permeable material, probably gravel, in the vicinity of test point G-4. It is most probable that the deposit is the remains of the old creek bed. The extent of this deposit in a direction parallel to the dam does not probably exceed a distance of 45 feet and is most probably not deeper than a depth of 35 feet relative to ground level at G-4.

The resistivity measurements at G-4 showed the highest permeability of all measurements in the area. Also, the available data from holes previously drilled indicated that most of the asphalt was injected into the dam at a considerable distance north of G-4, which could possibly

explain its failure to plug the leak.

This data, of course, represents the sub-structure at the toe of the dam. As to the course of the gravel deposit beneath the dam one cannot conclude from this data.

Richard D. Rechtien

Richard D. Rechtien

19 October 1967

TO: Brucker & Thacker
Consulting Engineers
1045 S. Brentwood Blvd.
Richmond Heights 17, Missouri

FROM: Dr. Richard D. Rechtien
Geophysical Consultant
4 Crestvied Ave.
Rolla, Missouri 65401

SUBJECT: REPORT OF GEOPHYSICAL STUDIES ON THE UPSTREAM SIDE OF LAKE WAUWANOKA DAM, HILLSBORO, MISSOURI

INVESTIGATION OBJECTIVE:

After sealing of the leak in the Lake Wauwanoka Dam in late September, additional geophysical studies were requested on the upstream face of the dam, which had been inaccessible in the previous investigation. The purpose of the present study was to detect any horizontal discontinuity in the rock structure underlying the upstream face of the dam and to determine any areas of high permeability, which in turn would indicate a possible avenue of water leakage beneath the dam.

FIELD INVESTIGATION:

The electrical resistivity method was again employed to investigate the subsurface. Field work was conducted on the 30th of September and the 1st of October 1967.

Vertical profile data was taken at sixteen equally spaced locations along a line parallel to the water's edge. The elevation of the line of traverse, which was approximately constant, was 64 feet relative to your survey. The interval between stations was 30 feet; test point G18 corresponding to the iron stake surveyed at approximately 496 feet relative to your drawing.

Data was taken to depths of 25 feet with the exception of test points G15 and G24, where the depths of investigation were 45 and 41 feet respectively. The 25 foot depth was considered adequate for a satisfactory determination of bedrock conditions.

RESULTS:

The results of this investigation are summarized in Figures 1 and 2.

Figure 1 is a direct overlay of the dam profile figure supplied by G. Brucker. This figure presents the profile of the underlying rock structure as interpreted from the data.

The overburden-bedrock interface was detected at approximately 10 feet at station G28 on the north end of the dam face and decreased in elevation to approximately 20 feet at station G19. Bedrock was again detected at 20 feet in depth at station G14 and became shallower southward. Between station G15 and G16 the occurance of bedrock at a depth of 20 feet was indicated (as shown in Figure 1 be a dotted line), however the data was not sufficiently definitive at these points to reach a definite conclusion for this depth.

At a depth of 24 feet, for all measuring points, a strong electrical resistivity variation was detected which indicated a firm, continuous rock layer underlying the dam along this line of traverse. The depth profiles were terminated at this point with the exception of the data from test points G15 and G24. At these locations data from deeper layers was obtained for correlation with data from the previous survey. The horizontal lines on the figure define strong resistivity variations and indicate the interface between rocks of different electrical properties.

Figure 2 shows the horizontal variations in the electrical resistivity of the rocks at depths of 15 and 20 feet. The apparent electrical resistivity, as plotted, is inversely proportional to the permeability and water content of the subsurface material. The higher apparent electrical resistivity at a depth of 20 feet as compared to that at 15 feet reflects the decrease in permeability with depth.

The low point in either curve indicates the area of high permeability and moisture content. This point occurs in both cases at station G15. However, this variation as here shown reflects the discontinuity of the shallow rock layers at these points as shown in Figure 1, where a lower resistive soil has replaced the higher resistive rock layers. There is no indication here that a higher than normal permeability persists in this area at depths greater than 24 feet.

CONCLUSIONS:

Both Figures 1 and 2 indicate the presence of the old stream channel in the vicinity of test point G15. However the total variation in electrical resistivity across the dam face does not indicate the presence of a possible avenue of leakage. The underlying rocks on the upstream side of the dam appears to be in much better condition than those on the back side.

The results of this investigation do not show any regions of significant seepage.

Richard D. Rechtien

UNITED STATES DEPARTMENT OF AGRICULTURE

SOIL CONSERVATION SERVICE

1021 North Kirkwood Road, Kirkwood, Missouri 63122

April 12, 1974

Mr. George Kristof
3852 Virginia
St. Louis, Missouri

Dear Mr. Kristof:

Following is my report of recommended solutions to the problems I surveyed with you on April 9th concerning the Lake Wauwanoka development area.

The present dam shows some wave action damage on the upper side. If the water line is to be maintained at its present level I would suggest some additional rock rip-rapping which should extend approximately one (1) foot above your permanent water line. There is some wave action damage along the shore line, however, this is not showing evidence of creating a serious problem at this time. It would be beneficial to rip-rap these areas or build a rock retaining wall like some of the other land owners have done to protect the shore line. There is some minor damage on the back side of the dam that needs reseeding where motor bikes have been using the area as a path. Pathways on the back side should be prohibited because they tend to kill the grass and with the bare soil exposed the water would concentrate in that area and a serious erosion problem could occur. These areas need to be torn up and reseeded as soon as possible before some erosion does start.

The brush on the back side of the dam should be eliminated. Where you have sprayed, it appears like the brush is dead. You may want to try burning the dead debris that is present and reseed the back of the dam with K31 fescue - about 100 lbs. You may not get much germination at this time of the year by broadcasting it in the heavy mulch cover that you have. I would recommend broadcasting more seed during the months of January and February to try to improve the stand of grass. You can also sprig some crown vetch on the back side. If you do not desire to sprig the entire area at one time you can start a small patch and then use this as a source for future sprigs.

There is some seepage evident on the back tow of the fill, however, it doesn't appear to be serious at this time. If you desire this could be intercepted and dried up with an underground perforated pipe laid along the back side of your dam. This would also tend to dry up the proposed location of road along the back side of the dam. If a road is installed back there I think some type of drainage pipe is needed to permit a solid base for a roadbed. The emergency spillway is built on solid rock and does not appear to be creating much damage. I would

Chart 4-1

caution you to be aware of any erosion activity along the banks of the spillway and if some should occur then this needs to be rip-rapped with heavy rock to protect the banks from caving off.

A concrete box could be installed in the spillway to permit you to regulate the flow of water and for this I would request our engineer to provide a design for you. I would not recommend any blasting in that area because this could possibly cause a fissure in the rock and open up another crack that would permit additional seepage from the pond.

A small pond could be constructed in the valley below the present lake and this could be used as a rearing pond for fish production. I would recommend that all of the area below the lake be used for recreation type of activity. No permanent homes should be constructed in the flood plain below the dam because of the possible loss of life if the dam should fail.

The road surrounding the lake is located ideally to serve as a diversion around the lake to intercept the hillside water and to help reduce siltation to the lake. The road ditch on the upper side of the road needs to be cleaned out and maintained so it will act as a diversion. The road ditch needs to have positive drainage so it doesn't hold water that would saturate your roadbed and which, in turn, would break the road up by creating soft road conditions.

The main areas to watch from this road are the culverts under the road and where they empty into the natural drainage ditches on the lower side of the road. These could become active but at the present time they are on rock and do not appear to be real active. A rock dam could be built across these small drainage ditches to slow the water down and to collect silt. This rock dam should be about three (3) feet high. It can be placed in the ditch with the base about four (4) foot wide and tapering at the top to approximately two (2) feet in width. The center of the rock dam needs to be lower than the two ends to permit the water to go over the rock and not around the end of the dam. These could be placed periodically throughout all of the drainage ditches that you have.

The spring along Hillsboro Creek could probably be cleaned out to increase its flow although it isn't assured the flow will be increased by cleaning it out but it does show evidence of some silt building up around the point where the spring is trying to seep out and it is forcing some water out in a different location so it may be possible that if some of this silt is cleaned out the spring may flow heavier.

There is another scenic spring on up the creek about a mile that has a rock wall around it that could be cleaned out and rebuilt which would serve as a nice scenic development area, a bird sanctuary, and a picnic

area established there if the people in the development so desired.

X I observed a pfitzer in the south side of the circle that is quite old and has lost its lower limbs. It may be desirable to remove it and replant with a new one to maintain the beauty of the area. Some of the pine trees at the entrance way show some frost damage. If the buds are dead, that tip of the branch will not continue to grow, although after a bit of time the other limbs will probably fill around it. I do not think that the trees will die from the light frost damage. They are getting to be a large size and are blocking the view as you enter the lake development. Some interest was shown in removing these and planting some more of smaller size. If this is done I would recommend the trees be planted in a wider spacing so when they do mature you will not be confronted with the same problem again.

XI There has been a new pond constructed on the northeast fork which is across your property line. This pond is serving as a silt basin for you and it is a good practice to build these on draws above your establishment although this pond has no pipe in it to handle the excess drainage it is receiving and the spillway may tend to wash out. If this happens he will need to repair the spillway or he may wish to install a pipe. Overall your drainage into the lake is of an excellent nature. It is predominantly woodland with a good leaf mulch cover and underbrush that is protecting the land and is not producing much silt, as is evident by looking in the coves and the other drainage ditches going into the lake. There is a small area on the top of the ridge as you enter the lake that is seeded in good fescue. There is not an excessive amount of erosion and I would not anticipate any as long as the land is used as it is at present.

XII The timber that you propose to cut will probably result in some damage because of the necessity to build a logging trail to the area. I would recommend that all the disturbed area be immediately seeded with perennial rye and K31 fescue to provide a quick cover to help reduce any siltation of the area.

The major stream coming from the west into the lake does not show much activity cutting from the banks. There is some minor erosion in the curves of this creek, these need to be watched and if they continue to be active then these curves should be rip-rapped with heavy rock. There was one point in the curve where the creek water was jumping out of its bank at flood time and is running down the dirt road creating quite a scour problem. The low spot in the creek on the east bank should be filled up to eliminate the water jumping out of its bank at that location.

XIV If I can be of any further assistance to you please feel free to call.

Sincerely,

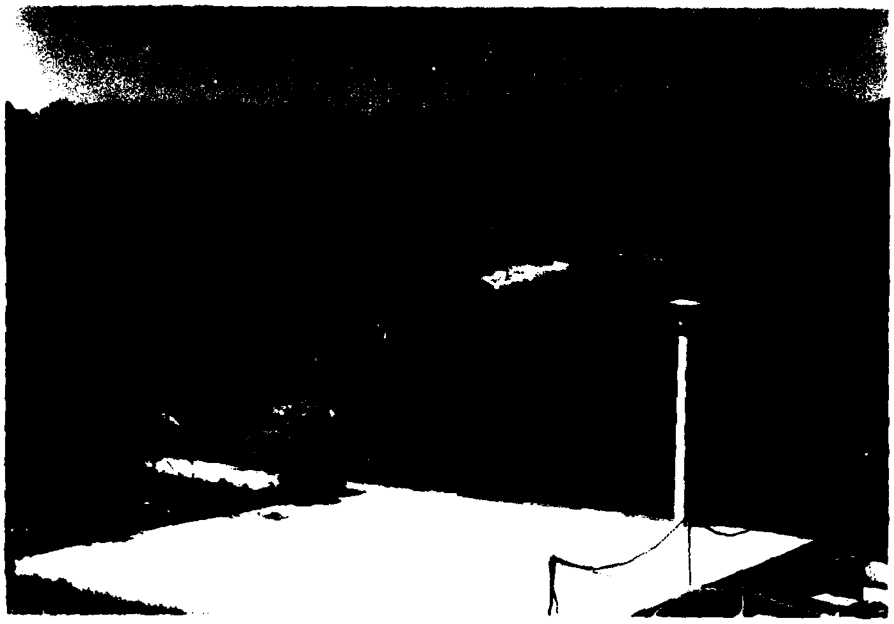
Richard L. McMillen

Richard L. McMillen
District Conservationist

Chart 4-3

cc - Max Mull, AC

APPENDIX



NO. 1: UPSTREAM FACE OF DAM

NO. 2: DOWNSTREAM FACE OF DAM

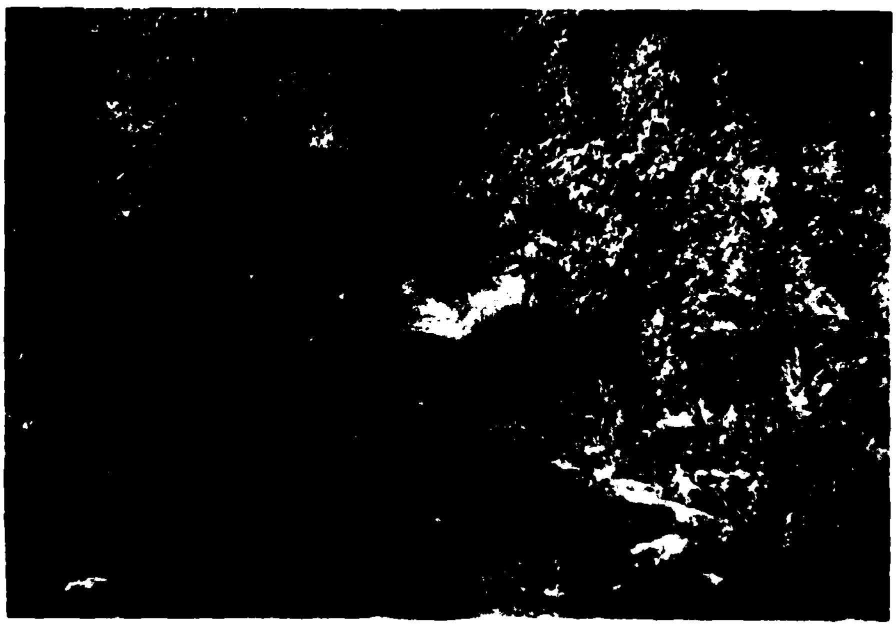
NO. 3: BROKEN SUBDRAIN PIPE

NO. 4: SUBDRAIN OUTLET

NO. 5: PONDED SEEPAGE

NO. 6: CATTAILS BELOW DAM

NO. 7: SPILLWAY WEIR AT DAM



NO. 8: SPILLWAY EXIT CHANNEL

HYDROLOGIC COMPUTATIONS

1. The HEC-1 Dam Safety Version (July 1978) program was used to develop inflow and outflow hydrographs and dam overtopping analyses, with hydrologic inputs as follows:

a. Probable maximum precipitation (200 sq. mile, 24-hour value equals 25.7 inches) from Hydrometeorological Report No. 33. One hundred year frequency (one square mile precipitation, 24-hour value equals 7.22 inches) from U.S. Weather Bureau Technical Paper No. 40.

b. Drainage area = 2.06 square miles
= 1,320 acres

c. SCS parameters
Lag time = 0.25 hours
Soil type CN = 91

2. The spillway section consists of a broad-crested, approximately U-shaped concrete and rock section for which conventional weir formulas do not apply.

Spillway release rates were determined as follows:

(1) Spillway crest section properties (area, a and top width, t) were computed for various depths, d.

(2) It was assumed that flow leaving the spillway crest would occur at critical depth. Flow at critical depth (Q_c) was computed as $Q_c = \left(\frac{a^3}{t} g\right)^{0.5}$ for the various depth, d.

Corresponding velocities (v_c) and velocity heads (H_{vc}) were determined using conventional formulas.

(3) Static lake levels corresponding to the various Q_c values passing over the spillway were computed as critical depths plus critical velocity head ($d_c + H_{vc}$), and the relationship between lake level and spillway discharge was thus obtained. The procedure neglects the minor insignificant friction losses across the length of the spillway.

3. The profile of the dam crest is irregular and flow over the dam crest cannot be determined by conventional weir formulas. Flow quantities overtopping the dam crest were computed as described in the preceding paragraph and corresponding flow over the dam and spillway for given elevations were added to obtain the combined outflow rating curve for the dam and spillway. This rating curve is shown on Plate 10. The inflow and outflow hydrographs for the PMF are shown on Plate 11.

```
**********************************
FLOOD HYDROGRAPH PACKAGE (HEC-1)
DAM SAFETY VERSION      JULY 1978
  LAST MODIFICATION  3 AUG 78
**********************************
A1       ANALYSIS OF DAM OVERTOPPING USING RATIOS OF PMF
A2       HYDROLOGIC-HYDRAULIC ANALYSIS OF SAFETY OF LAKE WAUWANOKA DAM
A3       RATIOS OF PMF ROUTED THROUGH RESERVOIR
B      288       0       5      -0      -0      -0      -0      -0      -0      -0
B1       5
J        1       3       1
J1    0.27    0.50    1.00
K        0  INFLOW                               3       1
K1        INFLOW HYDROGRAPH
M        1       2    2.06                     1.0                               1
P        0    25.7     102     120     130
T                                                     -1     -91
W2            0.25
X     -1.0    -.10     2.0
K        1     DAM                       2       3       1
K1       RESERVOIR ROUTING BY MODIFIED PULS
Y                                1       1
Y1       1                                         2370      -1
Y4     612     613     614     615     616   616.6     617   617.5     618   618.5
Y4     619   619.5     620     621     622
Y5       0     150     490     940    1600    2000    2390    3300    4590    6300
Y5    8740   11630   14920   22560   31450
$A       0    86.3   102.8   171.7
$E   529.6     612     620     640
$S   612.0
$D   616.6
K       99
```

```
                 ***************
         ... A KAGE (HEC-1)
         ...        JULY 1978
         ...      3 AUG 78
****************************************
          A1        ANALYSIS OF DAM OVERTOPPING USING 100 YR FLOOD
          A2        HYDROLOGIC-HYDRAULIC ANALYSIS OF SAFETY OF LAKE WAUWANOKA DAM
          A3        100 YR FLOOD ROUTED THROUGH RESERVOIR
          B     288       0       5      -0      -0      -0      -0      -0      -0      -0
          B1      5
          I       1       1       1
          J1    1.0
          K       0  INFLOW                              3       1
          K1        INFLOW HYDROGRAPH
          M       0       2    2.06                    1.0                       1
          O     288
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .014    .014    .014    .014    .014    .014    .014    .014
          01   .014    .014    .014    .014    .014    .014    .014    .014    .014    .014
          01   .014    .014    .014    .014    .014    .014    .014    .014    .014    .014
          01   .014    .014    .014    .014    .014    .014    .014    .014    .022    .022
          01   .022    .022    .022    .022    .022    .022    .022    .022    .022    .022
          01   .022    .022    .022    .022    .022    .022    .031    .031    .031    .031
          01   .031    .031    .061    .061    .061    .061    .061    .061    .132    .132
          01   .133    .251    .251    .550    .823    .382    .251    .133    .132    .132
          01   .061    .061    .061    .061    .061    .061    .031    .031    .031    .031
          01   .031    .031    .022    .022    .022    .022    .022    .022    .022    .022
          01   .022    .022    .022    .022    .022    .022    .022    .022    .022    .022
          01   .014    .014    .014    .014    .014    .014    .014    .014    .014    .014
          01   .014    .014    .014    .014    .014    .014    .014    .014    .014    .014
          01   .014    .014    .014    .014    .014    .014    .014    .014    .014    .014
          01   .014    .014    .014    .014    .014    .014    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007    .007    .007
          01   .007    .007    .007    .007    .007    .007    .007    .007
          T                                                     -1     -91
          W2           0.25
```

```
W2              0.25
X      -1.0     -.10        2.0
K         1      DAM                                 2           3            1
K1           RESERVOIR ROUTING BY MODIFIED PULS
Y                                          1         1
Y1        1                                                                 2370         -1
Y4      612      613        614          615       616       616.6           617      617.5        618       618.5
Y4      619    619.5        620          621       622
Y5        0      150        490          940      1600        2000          2390       3300       4590        6300

Y5     8740    11630      14920        22560     31450
$A        0     86.3      102.8        171.7
$E    529.6      612        620          640
$$    612.0
$D    616.6
K        99
```